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DRAINAGE AND FLOOD-CONTROL ENGINEERING

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DRAINAGE AND FLOOD-CONTROL ENGINEERING

BY

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PREFACE

This book has been written primarily as a textbook for courses in land reclamation by drainage. It treats not only of the improvement of small areas of cultivable land by underdrainage, but also of the reclamation of large areas of wet and overflow lands by surface drainage and by flood control. The term flood control has been restricted by some to the protection of cities from flood damage, but, as a matter of fact, by far the larger part of the flood-control work which has been done has had for its chief object the protection of agricultural lands from overflow.

There has been a marked advance in drainage practice in recent years. Whereas, formerly, rules of thumb were largely used in planning ditches and tile drains, now the designs of drainage systems are based on the fundamental sciences of hydrology, hydraulics, and soil physics; and it is essential that the drainage engineer have a proper understanding of these sciences in so far as they affect land drainage. These sciences have been treated in the chapters on precipitation, flood run-off, flow of water in open channels, flow of water in tile drains, and soil physics, the treatment being from the specialized viewpoint of the drainage and flood-control engineer rather than general.

An attempt has been made to embody in the present volume a brief review of the results of recent researches in hydrology and hydraulics as affecting drainage and flood-control engineering. No claim is made for exhaustive treatment of these subjects, but it is believed that the material presented will serve in the ordinary work of the drainage engineer.

The chapter on excavating machinery has been included because it is believed that a knowledge of the adaptability, efficiency, and limitations of the various kinds of excavating machinery will prove of value to the engineer in the preparation of his design, in the writing of specifications, and in the judging of the adequacy of construction equipment.

The legal procedure involved in the formation of an organization for carrying out the plans of drainage is so important, and so

intimately related to the work of the engineer, that a chapter has been devoted to the subject of drainage law. The three phases of this subject which have been given special consideration are the organization procedure, the spreading of assessments, and the determination of damages.

In the preparation of this book, the author has drawn freely from the scattered literature upon the various topics. Where material has been reproduced from other books, due credit has been given. The author is especially indebted to the Bureau of Public Roads, United States Department of Agriculture, for its publications on drainage, and to the Miami Conservancy District for its *Technical Reports*.

The author wishes to acknowledge his indebtedness to Prof. C. C. Williams, Head of the Department of Civil Engineering, University of Illinois, and to Hardy Cross, Professor of Structural Engineering, University of Illinois, who read the manuscript and made many valuable suggestions.

GEORGE W. PICKELS.

URBANA, ILLINOIS.
September, 1925.

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DRAINAGE AND FLOOD-CONTROL ENGINEERING

CHAPTER I

INTRODUCTION

Land drainage has been one of the principal factors in increasing the wealth and security of nations. Without the large areas of lands which drainage and irrigation have made productive, it would not be possible to feed the present population of the earth.

Fertility of the soil is considered the most valuable of all natural resources, since, unlike coal, oil, and other minerals, it is practically inexhaustible if proper methods of cultivation are employed. But fertility of soil alone is not sufficient to produce crops. Air and the proper amount of water are equally essential, and these are supplied by removing the gravitational water in the top 3 or 4 feet of soil, thus permitting the air to enter the pore spaces in the soil and providing room for good root development.

During the past 75 years in the United States, over 53,000,000 acres of land have been rendered more productive by drainage. A large part of this area was practically worthless in its original state. Records of the Government Land Office show that at one time approximately one-fourth of the State of Illinois—and a still larger portion of other states—was swamp land. Assuming that the average annual return from these lands during the 75-year period has been $15 an acre, the total wealth produced from them has been about $30,000,000,000—an amount almost equal to the present valuation of the railroads and public utilities of the United States; and the present annual income from these lands is approximately $1,300,000,000. Although these amounts are necessarily approximate, they at least show that land drainage has played a very important part in the economic growth of the country.

At the present time, the food supply in the United States is greater than the demand, but with our population increasing at the rate of about 1,500,000 a year, the time is not far distant when the domestic demand will exceed the present supply. Fortunately, there are approximately 95,000,000 acres of swamp, overflowed, and tidal marsh lands yet to be reclaimed by drainage, and a large amount of arid lands which can be reclaimed by irrigation. Furthermore, the present arable lands can be made more productive by more complete drainage and more intense cultivation. Hence the field of the drainage engineer is a large one, and his work will become increasingly important with time.

Development of Land Drainage.—As the demand for agricultural lands increased, the market value of the land increased proportionally, and the potential value of the wet lands made their reclamation economically feasible. In general, the low, wet lands were more fertile than the high lands, and the larger crops from the reclaimed land were an additional inducement toward further reclamation. At first, only small, scattered areas were drained, the work being done by the individual owners. Later, larger areas were drained either by companies operating under government charter or by groups of individuals associated in drainage districts, operating under special drainage laws.

The first drainage was accomplished by small, open ditches, and a large aggregate area of wet, unproductive land was rendered capable of cultivation in this way. It was not until the invention of power machinery, especially the dipper dredge, that land reclamation was attempted on a large scale. It was found, however, that although ditches enabled the land to be cultivated, they did not lower the ground water fast enough after rains to permit the land to produce the maximum yields of which it was capable, considering its fertility. This fact was recognized as applying to the higher as well as to the lower lands. This difficulty could be overcome by constructing more ditches, but when placed close enough to give thorough drainage, the ditches occupied too large a proportion of the land area. The solution of this problem was the introduction of small covered drains emptying into the open ditches, thus permitting the land over the drains to be cultivated. The first underdrains were constructed of coarse rock or gravel, or fagots of brush laid in an open trench and then covered with topsoil. These drains became clogged with earth in a short time and were not satisfactory.

Conduits of clay were then employed—the forerunner of our present drain tile. These conduits were made by hand in two pieces, a flat base plate and an U-shaped upper section, the latter resting upon the former. This type of tile drain was standard in England from about 1810 to 1840. Naturally they were used to a limited extent only, because of the cost of manufacture. More recently, the invention of the proper machinery has made it possible for cylindrical tile of both clay and concrete to be manufactured in large quantities at a reasonable cost. The real era of underdrainage began with this invention, and today millions of acres of land are being rendered more productive by the use of tile drains.

Drainage in Europe.—The largest area of wet land in England which has been made productive by drainage is the strip of land along the east coast known as the Fens. This area of about 680,000 acres was at one time periodically overflowed by the tides of the North Sea, but is now a prosperous agricultural region and the chief source of grain supply in England. The reclamation of the Fens was accomplished during a period of several centuries under rather discouraging conditions. The land is protected from the sea by dykes or levees, and the interior drainage is effected through numerous ditches leading to the dykes where the water is raised and discharged by pumps.

Another well-known reclamation project is that carried out by the government of Holland. In 1839, the government undertook the drainage of Haarlem Lake, the bottom of which was below sea level. This lake, which was about 15 miles long, 8 miles wide, and 13 feet deep, and covered about 44,000 acres, was emptied of its water by large pumps after it had been surrounded by a dyke to keep out the sea. This work was completed in 1848 and today the bottom of the former lake is under cultivation and provides homes for some 18,000 people. Three large pumps remove the interior drainage water.

Several large projects have been carried out in France, the most notable of which are the La Gironde project, containing about 1,500,000 acres,[1] and the Forez project, containing about 140,000 acres.[1]

In addition to the large projects mentioned, whose works consist of open ditches and pumping plants, a large amount of underdrainage has been done, especially in France and Germany.

[1] ELLIOTT, C. G., "Engineering for Land Drainage," p. 14.

Drainage History in the United States.—Land drainage by small, open ditches was probably practiced to some extent in the seaboard states at an early date, although there are no definite records of such work until the early part of the nineteenth century. Underdrainage was practiced to a very limited extent. The first record of such work was in 1835 when John Johnson of Geneva, New York, started underdrainage on his farm. During the following 15 years he laid 16 miles of tile drains. A machine for making tile was invented in England in 1843 and one of them was imported to the United States in 1848.

The real era of land drainage began in this country about 1850 when Congress passed the Swamp Act. Under this Act, those portions of the Public Lands which in their natural state were too wet to cultivate were given to the states in which they lay with the understanding that the several states were to drain and sell them. One of the most debated subjects in Congress during the years 1848 to 1850 was the disposition of the swamp lands. Some contended that the government should undertake the reclamation of this land; others held the opinion that such an enterprise was too expensive for the government to engage in advantageously. A compromise was reached in the passage of the Swamp Act, which simply passed the question to the several states for solution. Some of the states, in turn, relieved themselves of the responsibility by giving the lands to the counties with the understanding that the counties were to drain the land. Some counties took their responsibility seriously and began the construction of county drains paid for from the taxes. In most instances, however, the land was sold to individuals in its original condition. Nevertheless, most of the states realized that under the common law only a small amount of drainage was possible. To assist in the work of reclamation, statute laws have been passed from time to time by all the states, whereby a majority of landowners might combine for constructing the ditches or drains needed for mutual drainage and force an unwilling minority to join in the enterprise and to pay its share of the cost. To illustrate the desire of the states to assist in this work, it may be stated that there has not been a single session of the Illinois Legislature in the past 50 years without the passage of one or more acts or amendments to the drainage law, to the end that the landowners might more easily reclaim their lands.

As the result of these laws, a large amount of land reclamation has been accomplished. Table I, abstracted from the drainage

TABLE I.—STATUS OF LAND DRAINAGE IN THE UNITED STATES, AS OF 1920
(United States Census of 1920)

State	Area, acres	Land in farms				Unreclaimed[1] swamp and over-flowed lands, acres
		Total, acres	Improved, acres	Provided with drainage, acres	Needing drainage, acres	
United States	1,903,215,360	955,883,715	503,073,007	53,024,975	39,110,357	95,074,000
Alabama	32,818,560	19,576,856	9,893,407	415,293	1,610,656	2,604,000
Arizona	72,838,400	5,802,126	712,803	9,651	41,951	
Arkansas	33,616,000	17,456,750	9,210,556	497,489	1,642,403	3,720,000
California	99,617,280	29,365,667	11,878,339	813,960	286,320	
Colorado	66,341,120	24,462,014	7,744,757	127,037	270,997	
Connecticut	3,084,800	1,898,980	701,086	14,646	56,462	96,000
Delaware	1,257,600	944,511	653,052	185,831	68,969	148,000
Florida	35,111,040	6,046,691	2,297,271	147,940	687,021	17,900,000
Georgia	37,584,000	25,441,061	13,055,209	274,688	1,819,611	7,102,000
Idaho	53,346,560	8,375,873	4,511,680	64,648	199,874	
Illinois	35,867,520	31,974,775	27,294,533	11,247,637	1,228,769	1,500,000
Indiana	23,068,800	21,063,332	16,680,212	8,308,844	1,717,068	1,575,000
Iowa	35,575,040	33,474,896	28,606,951	7,334,404	2,052,942	746,000
Kansas	52,335,360	45,425,179	30,600,760	106,985	68,292	975,000
Kentucky	25,715,840	21,612,772	13,975,746	225,228	573,299	1,365,000
Louisiana	29,061,760	10,019,822	5,626,226	1,004,935	1,095,769	7,500,000
Maine	19,132,800	5,425,968	1,977,329	26,302	142,055	1,210,000
Maryland	6,362,240	4,757,999	3,136,728	249,799	184,820	480,000
Massachusetts	5,144,960	2,494,477	908,834	39,022	80,883	175,000
Michigan	36,787,200	19,032,961	12,925,521	3,156,632	2,070,387	5,350,000
Minnesota	51,749,120	30,221,758	21,481,710	2,993,034	3,504,574	6,420,000
Mississippi	29,671,680	18,196,979	9,325,677	825,878	1,455,534	4,438,000
Missouri	43,985,280	34,774,679	24,832,966	859,663	830,693	3,535,000
Montana	93,523,840	35,070,656	11,007,278	51,146	113,293	
Nebraska	49,157,120	42,225,475	23,109,624	214,428	145,818	755,000
Nevada	70,285,440	2,357,163	594,741	46,252	59,739	
New Hampshire	5,779,840	2,603,806	702,902	11,777	40,783	85,000
New Jersey	4,808,960	2,282,585	1,555,607	174,260	77,881	695,000
New Mexico	78,401,920	24,409,633	1,717,224	47,311	49,102	
New York	30,498,560	20,632,803	13,158,781	1,180,423	720,183	925,000
North Carolina	31,193,600	20,021,736	8,198,409	1,066,933	1,925,343	5,130,000
North Dakota	44,917,120	36,214,751	24,563,178	89,054	211,305	1,100,000
Ohio	26,073,600	23,515,888	18,542,353	7,365,532	2,014,889	940,000
Oklahoma	44,424,960	31,951,934	18,125,321	107,014	265,786	870,000
Oregon	61,188,480	13,542,318	4,913,851	229,582	471,396	
Pennsylvania	28,692,480	17,657,513	11,847,719	318,955	554,690	225,000
Rhode Island	682,880	331,600	132,855	2,403	8,174	29,000
South Carolina	19,516,800	12,426,675	6,184,159	676,152	1,341,903	4,910,000
South Dakota	49,195,520	34,636,491	18,199,250	161,371	446,915	540,000
Tennessee	26,679,680	19,510,856	11,185,302	254,118	640,479	996,000
Texas	167,934,720	114,020,621	31,227,503	756,263	4,130,614	5,100,000
Utah	52,597,760	5,050,410	1,715,380	74,316	165,926	
Vermont	5,839,360	4,235,811	1,691,595	35,649	68,912	104,000
Virginia	25,767,680	18,561,112	9,460,492	225,068	1,172,580	1,400,000
Washington	42,775,040	13,244,720	7,129,343	274,696	576,005	
West Virginia	15,374,080	9,569,790	5,520,308	38,464	310,868	531,000
Wisconsin	35,363,840	22,148,223	12,452,216	653,411	1,839,273	3,850,000
Wyoming	62,430,720	11,809,351	2,102,005	35,654	69,066	

[1] The values in this column were compiled by the Bureau of Public Roads, United States Department of Agriculture, in 1922. Of the total area given for the United States, about 56⅔ million acres are swamp and periodically swamp, 30⅔ million periodically overflowed, and 7⅔ million tidal marsh. The area covered by this compilation lies east of the one hundred third Meridian.

census of 1920, shows that of the 955,883,715 acres of land in farms in the United States 53,024,975 acres, or 5.5 per cent, have

been rendered more productive by artificial drainage. This drainage represents in most cases the result of work done by the farmowner, either independently or supplemental to the work done by organized drainage enterprises in providing outlet facilities; that is, it represents underdrainage as distinguished from open ditch or large tile outlets. A large portion of this work of underdrainage, however, was made possible by the open ditches or large tile drains constructed by organized drainage districts.

Table II shows the progress of land drainage in the United States up to 1920 through organized drainage enterprises. It will be noted that there was a marked increase in the amount of land reclamation during the period from 1880 to 1889, due to the invention of the dipper dredge in about 1885. Each year an increasing amount of work was done, until a maximum of 17,344,098 acres was reached in the 5-year period from 1910 to 1914. The smaller amount of reclamation accomplished during the succeeding 5 years was the result of the World War. Altogether a total of 65,495,038 acres have been drained to a greater or less extent through the work of organized enterprises at a total expenditure of $372,273,567. The work consists of 111,770 miles of open ditches, 45,174 miles of large tile drains, 4,330 miles of levees, and 453 pumps. The distribution of this work by states is shown in Table I.

Future Development of Land Drainage.—Table I also gives the amount of swamp, overflow, and tidal marsh lands which can be economically reclaimed by drainage.

It is difficult to estimate the field of underdrainage. Although 53,024,975 acres are given in the census report as having been made more productive by drainage, yet only a small percentage of this area is underdrained properly and thoroughly. The author is familiar with thousands of acres of land in Illinois which are said to be underdrained; but in most instances the drainage consists of only a few random lines of tile, serving the lower wet areas. There are comparatively few farms which have a complete scientific system of underdrainage. As the population increases, and with it the demand for farm products, the need and value of underdrainage will become more and more apparent and necessary. Even high land which has good surface drainage is much improved by underdrainage, as recent drainage work has proved.

TABLE II.—SHOWING PROGRESS OF LAND DRAINAGE IN THE UNITED STATES TO 1920 THROUGH ORGANIZED DRAINAGE ENTERPRISES

Date of organization	Acres	Capital invested, dollars	Ditches, miles	Tile, miles	Levees, miles
Before 1860.....	97,319	$ 182,716	173.7	4.2	
1860–1869.....	783,357	1,689,852	2,295.5	47.5	21.0
1870–1879.....	2,077,717	8,126,391	6,671.9	372.5	64.2
1880–1889.....	5,424,294	23,934,330	14,763.2	2,582.9	218.7
1890–1899.....	6,026,937	24,498,861	12,747.3	4,306.0	376.1
1900–1904.....	7,606,753	26,706,464	12,308.3	3,735.4	438.5
1905–1909.....	14,530,197	76,072,320	20,538.9	7,877.8	801.1
1910–1914.....	17,344,098	120,269,596	24,875.7	11,812.3	1,331.1
1915–1919.....	11,374,819	87,379,002	16,897.5	14,237.7	885.8
Not reported.....	229,547	3,414,035	498.0	198.4	193.5
Total.....	65,495,038	$372,273,567	111,770.0	45,173.8	4,330.0

It would be rash to prophesy just what percentage of the 955,863,715 acres of farm land in the United States will eventually be underdrained, but it is safe to say that the percentage will be large. Also, underdrainage in the future will be done on a more scientific basis than in the past, and the drainage engineer will need to be as familiar with soil physics and hydrology as he now is with surveying and hydraulics.

Flood Control.—The need for flood control is the result of man's attempt to appropriate to his own uses the bottom lands along the channels of streams—lands which are really part of the channels and which have been used periodically for centuries by the streams in discharging their flood waters. These areas are called the flood plains of the stream, and their widths are an indication of the magnitude of the flood flows which have occurred in the past and which will recur at greater or less intervals for all time to come, since the flood plains have been fashioned by the streams themselves to accommodate their flood flows.

The first settlements were along the rivers and smaller streams, but they were located on the bluffs and above the reach of the flood waters. As the settlements grew into villages and the villages into towns and finally into cities, the land along the waterfront became more and more desirable, especially for

railroads and industrial concerns, with the result that the flood plains were gradually encroached upon during the intervals between great floods; and in some cases even the channels themselves were narrowed by dumping, thus causing the water to flow at a higher stage and to overflow the banks of the streams at a smaller discharge than before. Railroad and highway bridges were constructed with openings inadequate to pass the flood waters, thus further restricting the flow. The result is that many of our large cities, located on rivers, have flood problems, and a new field has been created for the engineer.

The cities are not alone to blame in many instances. The farmers along the valleys have coveted the rich bottom lands for agricultural purposes and have joined together and constructed levees along the rivers' banks to confine the flood water and protect their bottom lands from overflow. Usually these levees have been constructed piecemeal and without coordination, each group working for its own interests without regard to the injury which its levees might cause to other lands, with the result that levees on opposite banks have been placed too close to each other, thus constricting the flood channel and backing up the water above.

While it is right for man to use the bottom lands because of their value for various purposes, he cannot expect thus to dispossess the streams without paying the price, either in the form of flood damages or in the form of assessments for the construction of flood protection or flood prevention works. As the country develops, the need for these bottom lands will become greater, and if other provisions can be made for the flood waters at a reasonable cost, there is no question that such provision will be made.

Flood Damages.—There are no valid reasons for believing that recent floods are of greater magnitude than those which have occurred in the past, opinions to the contrary notwithstanding, or that the frequency of great floods is increasing. The flood of 1844 on the Missouri, Illinois, and Upper Mississippi still holds the record of maximum flood discharge; and the flood of 1883 on the Ohio River has not been equaled since that time. In fact, few of the recent floods which have received such wide publicity are as large as former floods of record. Floods of large volume will continue to occur periodically for all time to come and man cannot prevent them, but he can and must control them, and

thus prevent the increasingly large losses of life and property caused by floods.

Although the flood discharges are not becoming greater, the flood heights are; and the more the river channels and flood plains are restricted, the greater will be the heights to which the flood waters will rise, and the greater the damage caused by flooding. Thus each year, the streams of the United States take a greater toll. This is illustrated in Table III, which gives the estimated flood losses for the years from 1900 to 1908.

TABLE III.—ESTIMATED FLOOD DAMAGES IN THE UNITED STATES[1]

Year	Amount of damage
1900	$ 45,675,000
1901	45,438,000
1902	55,201,000
1903	97,220,000
1904	78,841,000
1905	98,589,720
1906	73,124,000
1907	118,238,000
1908	237,860,000

[1] *Water Supply Paper* 234, United States Geological Survey, p. 25.

The Pittsburgh Flood Commission estimated the loss to the city of Pittsburgh, from 1898 to 1908, at $12,000,000, and the average annual loss at over $1,000,000.

The flood of March, 1913, in Ohio, caused a loss of over $100,000,000 and 360 lives in the Miami Valley, and of about $10,500,000 and 145 lives in the Scioto Valley. The property loss caused by this storm in the Ohio Valley was estimated at over $180,000,000.

More recently, the June 3 to 5, 1921, flood on the Arkansas River caused a property loss of over $19,000,000 and the loss of 150 lives at Pueblo and vicinity. On Sept. 9, of the same year, a flood on the San Antonio River caused a loss of from $5,000,000 to $10,000,000 of property and of 75 lives at San Antonio.

The above examples are only a few of the many floods which might be cited, and are given simply to point out the large amount of damage which is caused by floods. These losses are the direct losses caused by floods and do not include losses to railroads,

public utilities, manufacturers, merchants, and business men from suspension of business. Nor do they include the losses due to depreciation of property values. These intangible losses are difficult to estimate, but in the aggregate they amount to large sums.

Where the area flooded is agricultural lands, the damage is usually represented by the loss of whatever crops may be growing on the land and possible of stock. The land itself is seldom injured. In fact, the deposit of silt from the flood waters is usually beneficial. But in some instances the land is ruined either by the washing away of the topsoil or by the deposits of sand and gravel left by the flood. Near Pueblo, 4,700 acres of farming land were thus ruined for agriculture by the 1921 flood.

Status of Flood Control.—It generally requires, unfortunately, a specially disastrous flood to arouse the public and make it realize the need and value of flood protection, and to unite the various interests affected in an organization for the financing, planning, and construction of flood-control works. Thus it was the flood of Aug. 3, 1915, on Mill Creek which flooded 150 acres in the heart of Erie, Pa., killed 34 people, and caused $2,000,000 damage to property, which led to the construction of the 2-mile concrete conduit, 22 by 18 feet in cross-section, through the city.

Likewise it took the March, 1913, flood in Ohio to arouse the people of the Miami and Scioto valleys to the need for flood-control measures. The Miami Conservancy District spent about $36,000,000 in the construction of detention basins and in channel improvements. The Franklin County Conservancy District is spending $3,000,000 in channel improvement work at Columbus.

In California, flood-control works are being constructed on the Sacramento and Los Angeles rivers. In Colorado, plans are being prepared for flood relief at Pueblo. On numerous streams, large and small, throughout the country, channel improvement and levee construction are either being carried out or are being planned.

Future of Flood Control.—The importance and value of flood-relief measures are more fully appreciated than ever before. The works which have been constructed are concrete examples of what can be done to control floods, and will serve as models for other communities; and their success will prove an incentive to others to rid themselves of the flood menace.

Flood-control works are expensive, and their construction is warranted only when the value of the protection afforded by them is greater than their cost. Hence, it is only when the low areas are so necessary to the economic development of a city or rural community that they must be used for such utilitarian purposes, that the value of flood protection becomes great enough to justify the assessments necessary to pay for flood-control works; and even then the value of such protection must generally be demonstrated by actual flood losses.

The industrial and agricultural development in a large number of river valleys throughout the country have already reached the point where flood protection is justifiable on economic grounds, and future development will continue to add others to the list. Without doubt the flood-control movement, so ably started in recent years, will have a steady growth and will prove a remunerative field for the hydraulic engineer.

The protection of agricultural bottom lands in the larger river valleys by levees has been practiced for many years. Along the Mississippi River, levees were built as early as 1717, although the first ones constructed were only a few feet in height. As new levees were built, the flood stages became higher and it became necessary to increase the heights of those first constructed. The work of raising the levees to withstand the continually increasing flood stages is still in progress, and at the present time there are about 400 miles of levees between Cairo and the Gulf which are below the standard grade adopted by the Mississippi River Commission following the 1922 flood.

Along the Illinois River, 223,400 acres of bottom land are protected by levees. This area is about 67 per cent of the total area originally subject to overflow. Here, as elsewhere, the levee districts have had to raise their levees from time to time to meet the new flood heights. During the 1922 flood, 12 districts, aggregating 72,800 acres, were flooded, and the resulting damage exceeded $3,000,000.

Along the smaller streams, there are millions of acres of land which are flooded so frequently that little effort is made to cultivate them. A recent drainage survey in Illinois, made by the State Geological Survey, disclosed the fact that there are 1,136,700 acres of overflowed land within that state. Somewhat similar conditions exist in other states, making a large total area of overflowed lands to be reclaimed. In Illinois, flood protection is

to be secured by channel improvement and levee construction. Several stream-straightening projects have already been effected with satisfactory results, and one on the Sangamon River is now in process of construction (1925).

Flood-control projects should be subject to the approval of the state to safeguard the various interests which have a natural claim upon the water which is precipitated on their respective watersheds, and to see that the water is used to the best interest of all the people. The various interests involved are water supply, sanitation, drainage, irrigation, flood control, and possible navigation and waterpower development. Since these interests are more or less antagonistic, there should be some properly constituted authority to determine the rights of each. Some states have such officers or commissions whose approval must be obtained to all plans which will affect the natural flow of the streams. Where such authority exists, drainage and flood-control engineers should be familiar with the policies adopted by these departments, and should adhere to them as closely as possible to insure the prompt approval of their plans.

CHAPTER II

PRECIPITATION

In all engineering problems dealing with the storage, utilization, or removal of water, the amount of run-off from a watershed and the manner of its distribution are the chief factors upon which the engineering design is based. If the amount of water available, or the amount to be removed, is incorrectly estimated, or if the factor of safety used is not large enough to cover the uncertainty in the estimated amount, the project will be a failure and the reputation of the engineer responsible for the assumptions will suffer, even though the engineering structures are correctly designed for the conditions as assumed.

If run-off data were available for all watersheds, there should be no particular need for precipitation records. Unfortunately, records of run-off are available for comparatively few watersheds and these for short periods of time only. The engineer is compelled, therefore, to estimate the run-off largely from his knowledge of the precipitation. While there is no definite ratio of general application between rainfall and run-off, yet the latter is dependent on the former. A knowledge of precipitation, its amount, occurrence, and distribution, therefore, are of primary importance in flood control and drainage work.

Source of Precipitation.—The source of precipitation is the water which is evaporated from oceans, lakes, rivers, and other water surfaces, and from the soil, and that which is given off through the leaves of plants in the process of transpiration. Precipitation and evaporation are two events in a cycle which is continually taking place, and which will continue as long as the sun shines. The evaporation from large bodies of water is greater than the precipitation over those areas, and the excess is carried by air currents over the land, where it is augmented by the water vapor evaporated from the land areas, and is precipitated. The excess evaporation from the water areas is equal to the run-off from the land areas.

The amount of evaporation depends principally upon the temperature and the evaporation opportunity. The mean

annual evaporation from water surfaces in the United States ranges from 20 inches in Maine to 100 inches in parts of Arizona, New Mexico, and Nevada. The evaporation from land areas cannot be measured directly, except from very small plots, and is dependent upon so many variable factors that it is difficult to estimate. Meyer[1] estimates the mean annual evaporation and transpiration losses from representative watersheds as follows: Mississippi River, 22.1 inches; Minnesota River, 21.6 inches; Root River, Minnesota, 25.4 inches; Ohio River, 20.6 inches; Roanoke River, Virginia, 23.9 inches; Tombigbee River, Alabama, 31.2 inches; Colorado River, 25.8 inches; and Pit River, California, 9.9 inches.

While transpiration occurs only during the growing season, evaporation takes place throughout the year, even when the ground is frozen and covered with snow and ice.

Causes of Precipitation.—As evaporation takes place, the vapor is diffused in the air at the surface, and the moisture-laden air rises, since it is lighter and warmer than the drier, cooler air above it. As it rises, the atmospheric pressure about it becomes less and it expands. The heat used in the work of expansion is taken from the air itself, and as a result the air becomes cooler and its relative humidity greater. As it continues to rise, the air becomes cooler and cooler until its relative humidity is 100 per cent, that is, it is saturated with water vapor. Any further rising and cooling will cause some of the vapor to be condensed or changed to the liquid state in the form of minute particles of water. These particles are the substance of which clouds are composed. Gravity acting on the particles tends to pull them toward the earth; but this force is counteracted by the rising currents of air, so that instead of falling they are suspended in the atmosphere and carried along by the wind. When condensation is of sufficient extent, the cloud particles coalesce and form larger drops, which being too heavy to be held aloft by convection currents fall to the earth as rain.

The height to which moist air will rise depends upon its relative humidity and temperature. The dynamic cooling of unsaturated air quickly lowers its temperature, and it may become the same temperature as the surrounding air before the point of saturation is reached, in which case it ceases to rise. After condensation begins, because of the heat liberated in that process, the tempera-

[1] MEYER, A. F., "Elements of Hydrology," p. 428.

ture of the saturated air decreases more slowly than that of the surrounding air, and the air continues to rise, causing further condensation.

Although there are many theories as to the causes of condensation and precipitation, without doubt the most important and significant is that of dynamic cooling caused by the rising and expansion of vapor-laden air. Precipitation has been classified as convective, orographic, and cyclonic, based on the cause of the ascensional movement of the air.

Convective precipitation is limited largely to the equatorial regions where there is little or no wind, since these are low-pressure areas and the air movement is mostly vertical. Because of the high temperature, evaporation during the morning is rapid and of considerable extent. The vapor is carried upward by the convection currents and in the afternoon is precipitated. The same action takes place in the temperate zones in local thunder storms in connection with cyclonic precipitation.

Orographic precipitation occurs in mountainous regions. In order to pass over the mountains, the horizontal air currents are forced to rise, and this ascensional movement causes the moisture in the air to be precipitated on the windward slope. In the United States, orographic precipitation occurs on the westward slope of the Sierra Nevada and Rocky mountains, and on the eastward slope of the Southern Appalachian mountains.

Cyclonic precipitation is that which is experienced over the major portion of the United States. Here the ascensional movement which causes condensation and precipitation is the vertical component of the air currents in large areas of low pressure which move across the country from west to east.

Characteristics of Low-pressure Areas, or Cyclones.—The planetary winds in the latitudes of the United States are the prevailing westerlies; and at a distance of 5 or 6 miles above the earth's surface, the wind blows constantly from the west at a high velocity. Near the surface, however, the major movement is modified by minor disturbances which produce variable surface winds. The cause of this modification is the non-uniformity of the earth's surface; part of the area is water and part is land, and the land areas differ in topography and in physical and chemical composition, which results in unequal absorption of solar radiation. Since the air is heated by the earth's surface through radiation, conduction, and convection, the warmer the earth the

warmer the air above it will be, and the more rapidly will it rise. As the air over heated portions of the surface rises, the cooler, heavier air over surrounding cooler areas will rush in to take its place. This more or less horizontal movement of air currents is what is called wind. This movement is the result of changes in the atmospheric pressure. Over warm areas the atmospheric pressure is less than over cool areas, and is constantly changing as the result of temperature changes. It is just as natural for air to move from areas of high pressure to areas of low pressure as it is for water to flow downhill.

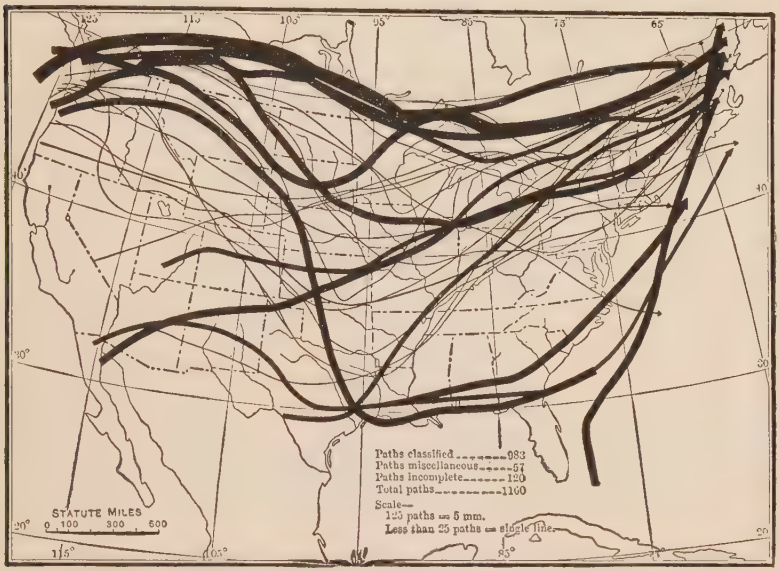
From causes not thoroughly understood, large areas of low pressure form in the larger air movements over the Pacific Ocean, the Rocky Mountain regions, the Gulf of Mexico, the South Atlantic Ocean, and during the summer months over the Central states, and are carried eastward by the prevailing westerlies. These areas are in the nature of vortices in the main stream of air, and are somewhat analogous to the whirls which form in streams, due to submerged obstructions in the channel, and move downstream in the current some distance before the whirling motion is dissipated.

Low-pressure areas are more or less elliptical in shape and vary in diameter from 500 to 1,500 miles. They follow each other across the country irregularly at average intervals of 3 or 4 days. Their average rate of movement is about 700 miles a day.

If the paths taken by all low-pressure areas which have been recorded were mapped, the entire United States would be covered with lines, and the impression gained from glancing at such a map would be that storms do not follow any definite paths. A careful study would show that there are certain routes which are taken much more frequently than others. These are shown in Fig. 1, and are worthy of careful study, since the amount, duration, and intensity of storms at any place are affected largely by its location with respect to these storm paths.

Low-pressure areas are characterized by rising temperature, falling atmospheric pressure, increasing cloudiness, precipitation, and changing winds. The atmospheric pressure at the center of these areas is normally about 29.5 inches of mercury, and at the circumference 30 inches. The air moves spirally inward and upward in a counterclockwise direction; hence, in the southern portion the wind is generally from the southwest; in the eastern portion, from the southeast; in the northern portion, from the

northeast; and in the western portion, from the northwest. The upward component of these winds causes the ascensional movement of the moist air necessary for condensation and precipitation.



(McAdie's "*Principles of Aerography.*")

FIG. 1.—Paths taken by storms in crossing the United States.

High-pressure Areas, or Anti-cyclones.—These are the reverse of low-pressure areas. They are characterized by falling temperature, rising barometer, light variable winds, and clear skies. They are much less definite and stable in form than low-pressure areas, and are more easily formed and dissipated. The atmospheric pressure at the center is normally about 30.5 inches of mercury.

Areas of low pressure and high pressure follow each other across the country, although the paths taken by the "highs" are more erratic than those taken by the "lows."

After precipitation has occurred and the low-pressure area has moved on, giving place to a high-pressure area, evaporation will take place rapidly, because of the evaporation opportunity and the low relative humidity of the descending air in the latter area. The moisture thus evaporated is carried by the air currents to the adjoining "lows," and re-precipitated.

Weather Maps.—The temperature, pressure, cloudiness, precipitation, and wind direction at points in the United States

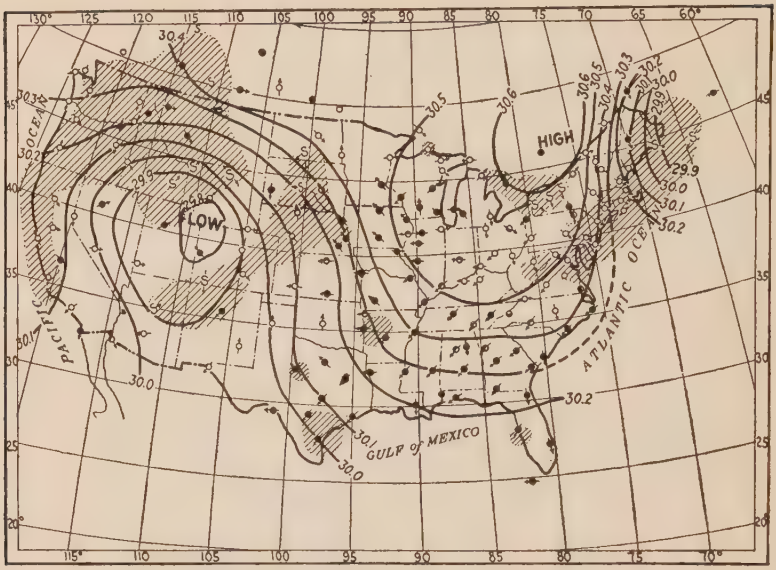


FIG. 2.—Weather map, Jan. 30, 1908.

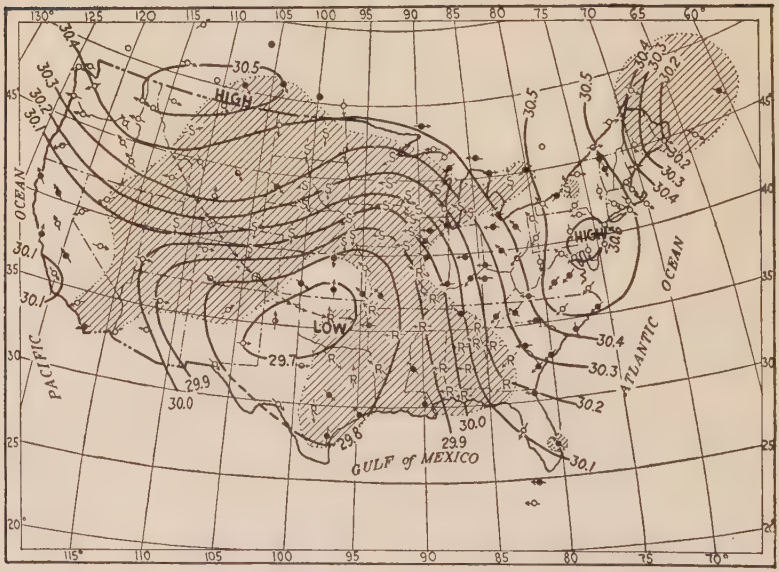


FIG. 3.—Weather map, Jan. 31, 1908.

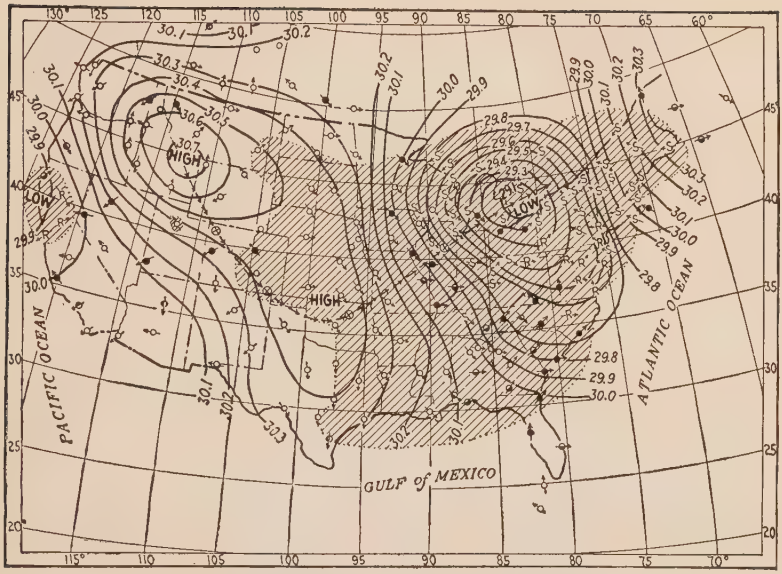


FIG. 4.—Weather map, Feb. 1, 1908.

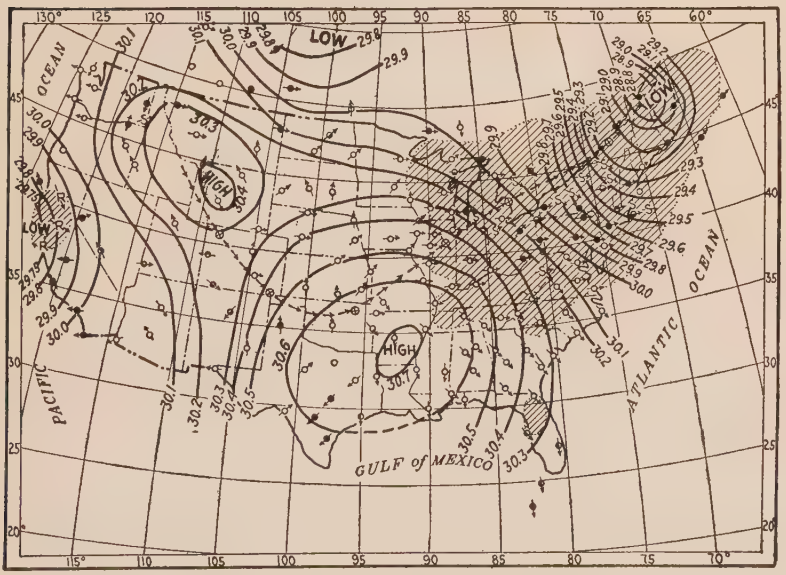


FIG. 5.—Weather map, Feb. 2, 1908.

are published daily in the form of maps by the United States Weather Bureau. The weather maps for Jan. 30 to Feb. 2, 1908, are reproduced in part in Figs. 2 to 5, and are typical of average weather conditions at that time of the year. The symbols on the weather maps have the following significance: The curved lines are isobars, or lines of equal atmospheric pressure; the shaded areas show regions of precipitation of 0.01 inch or more during the last 24 hours; the arrows point in the direction the wind is blowing; and the circles indicate the degree of cloudiness. This series of maps shows the progress of an area of low pressure across the United States. On Jan. 30, the "low" was centered over northwestern Colorado, and was moving toward the southeast. By Jan. 31 it had reached northern Oklahoma, and its direction was shifting to the northeast. On Feb. 1, it was centered over Michigan and had developed into a pronounced storm area, as indicated by the pressure of 29.1 inches of mercury at its center. On Feb. 2, its center was in Canada and it was following the St. Lawrence Valley to the Atlantic Ocean. The barometer was still falling and had reached a value of 28.85 inches at the center of the area.

Precipitation Records.—There are at present about 4,700 stations in the United States at which rainfall measurements are taken. Of these, about 200 are regular Weather Bureau stations where complete meteorological data are obtained. These stations are equipped with automatic rain gages which give a continuous record of precipitation, and are the only ones at which the precipitation during short periods of time is recorded. At the other stations, readings are taken once a day only.

Prior to 1849, the only records available are those which were kept by private individuals. Some of these date back to 1814 and are of considerable local value. In 1849, the Board of Regents of the Smithsonian Institute, realizing the practical as well as the scientific value of meteorological data, undertook the collection of such data, and about 400 stations were established through the cooperation of volunteer observers. In 1870, the government assumed control of this work, which was first placed in charge of the Chief Signal Officer of the United States Army and later transferred to the Weather Bureau which was organized by Congress in 1890. This Bureau installed many new stations, and knowledge of the amount and distribution of precipitation is, therefore, more complete subsequent to this date.

Naturally, the stations are more or less concentrated in the more densely settled portions of the country, and, therefore, the conclusions drawn from a study of the records in those portions are much more reliable than those for the more recently settled portions.

Annual Precipitation.—The mean annual precipitation in different parts of the United States is shown in Fig. 6. It will be observed that the largest amounts of precipitation occur over the western slope of the Sierra Nevada Mountains in the northwest portion of the United States, over the Rocky Mountains in Colorado, and over the Southern Appalachian Mountains in North Carolina. In all three instances, the precipitation is orographic. Most of the low-pressure areas enter the United States from the northwest, and the moisture-laden air from the Pacific, in rising to pass the mountains, precipitates most of its moisture on the western slope. After passing the mountains, the air descends and becomes warmer and drier; hence there is little precipitation until the Rocky Mountains are reached, where the air is forced upward again, causing precipitation, but in smaller quantities than in the former case, since the moisture content of the air is less. The Southern Appalachian Mountains are in the path of the tropical storms which originate in the region of the West Indies, and heavy precipitation occurs on the southeast slope. These storms are less frequent than those in the Northwest and therefore the annual precipitation is less.

The lines of equal rainfall in Fig. 6 approximately parallel the coast lines of the Pacific and Atlantic Oceans and the Gulf of Mexico. The precipitation along the coasts is greater than it is inland because of the high moisture content of the air from these large bodies of water. As the air moves inland it becomes drier, although augmented by the evaporation from the land areas, and the precipitation becomes less. Little moisture from the large water areas reaches the north-central part of the country, and the precipitation here is derived largely from the evaporation from land areas, and, consequently, is smaller in amount than along the coasts.

Those localities which lie in the paths most frequently taken by low-pressure areas receive more precipitation than those which are off those paths. The small precipitation in the Southwest is due to this cause.

The values shown on the rainfall map are the means for the entire period of record. The annual precipitation, however,

varies considerably from the mean; and the maximum and minimum values in a period of 100 years might vary as much as 50

FIG. 6.—Map of the United States showing mean annual precipitation.

per cent from the mean. The variations above the mean are generally larger than those below.

There is no evidence in any record—even in those extending over periods of from 80 to 100 years—that there is any progressive increase or decrease in the amount of precipitation. Although the annual precipitation at a station may be unusually high or low for a number of consecutive years, seeming to indicate a progressive increase or decrease in the amount of precipitation, yet when the record for a long period is considered, it is seen that no such tendency exists. Also dry years may be experienced at one station at the same time as wet years at another station in the same section of the country. Some authorities maintain that the drainage of swamps and the removal of forests have tended to decrease the precipitation, and that the irrigation of arid lands have had the opposite effect. The concensus of opinion of Meteorologists, however, is that these activities of man are too insignificant to have any observable effect upon rainfall.

While at a particular station there are periods of low rainfall followed by periods of high rainfall, the lengths of these periods are very irregular and do not follow any definite law; and the amount of rainfall in any year is more a matter of chance than it is the result of a well-defined law.

The longer the precipitation record at any station, the more reliable is the mean. From a study of 42 records of from 50 to 97 years in length, Mead[1] found that the average deviation of the mean of a 5-year period from the mean of the entire period was 16.85 per cent; for a 10-year period, 9.64 per cent; for a 20-year period, 4.77 per cent; for a 30-year period, 3.02 per cent; and for a 50-year period, 2.52 per cent.

There is frequently considerable variation in the annual precipitation at stations only a short distance apart, due to intensive storms of small area; but over a watershed as a whole, the variation from year to year is usually less than that at individual stations.

In flood protection and drainage work, the distribution of the annual precipitation is of more importance than the total amount. Also, it is the extreme rather than the average conditions which are of primary concern.

Monthly Precipitation.—The shorter the period considered, the greater is the variation in rainfall; therefore, there is a much greater variation in the amount of monthly rainfall than there is

[1] D. W. Mead, "Hydrology," p. 217.

in annual rainfall. The month is an arbitrary division of time which is not recognized by precipitation.

Since temperature is the most important factor in both evaporation and precipitation, it is natural to suppose that rainfall would be greater during the summer than during the winter, and in general this is true, especially where the range in seasonal temperature is large and where the winter temperature remains below the freezing point for several months. The exception to this rule is the Pacific coast region, where the maximum precipitation occurs during the colder months. This is because low-pressure areas are more frequent over this region during the winter, and the warmer, moisture-laden air from the ocean is cooled below its dew-point temperature by the cold surface of the mountain slopes, resulting in precipitation.

In the New England States, the lake region, the Ohio valley, the Gulf States, and the lower Mississippi valley, the variation in the average seasonal distribution is small. Along the South Atlantic coast, in the Northwest, and throughout the Great Plains, the rainfall is much greater during the summer than during the winter. As stated above, this condition is reversed along the Pacific coast.

The above statements are only general and apply to average conditions. The monthly distribution in individual years varies widely. As an extreme example, there are records which show that for a particular month the rainfall during the wettest year of record was less than that during the dryest year. A study of frequency curves of monthly precipitation shows that precipitation is below the average for more than half the time.

Excessive Precipitation.—In those fields of hydraulic engineering which deal with the storage of water, the engineer is concerned primarily with the total yield of a watershed and the variation in the annual amounts of precipitation. In some fields of engineering, the monthly distribution is of chief importance; and in others, it is the excessive precipitation which occurs during shorter intervals of time. It is in the latter class that the problems of flood control and land drainage fall.

The best criterion for judging future precipitation is the occurrence of precipitation in the past, and if reliable records were available for a period of several hundred years, it would be possible to make satisfactory predictions as to the maximum and minimum amounts of precipitation which might occur in the

future and the frequency of occurrence of a storm of given intensity. Unfortunately, few long records exist, and hence it is necessary to introduce a factor of safety in the assumptions which are made from the available records. In those portions of the United States which were first settled, precipitation records are longer and more numerous than in those portions more recently settled, and, therefore, predictions can be made with greater certainty in the eastern part of the country than in the western.

Meteorologists have given much study to the origin, size, and paths of storms, and information on the general characteristics of storms is fairly definite; but until recently little attention has been given to the duration, intensity, and distribution of precipitation at different points within storm areas.

Studies of Excessive Precipitation by the Miami Conservancy District.—In order to determine a basis for the engineering design of the structures for the protection of the Miami River Valley from floods, the engineering staff of the Miami Conservancy District, under the direction of A. E. Morgan, Chief Engineer, collected data of all storms of record within the United States up to 1914 in which the precipitation was excessive. Excessive precipitation was defined as follows:

1. Where the normal annual precipitation is 20 inches or more: (*a*) a 1-day rainfall at one station amounting to 10 per cent or more of the normal annual precipitation; or (*b*) a total rainfall at one station amounting to 15 per cent or more of the normal annual precipitation, regardless of the number of days in the period of excessive precipitation, so long as the average rainfall for the period is not less than 1 inch in 24 hours.

2. Where the normal annual rainfall is less than 20 inches, a total rainfall of 4 inches or more, regardless of the length of the period, so long as the average rainfall for the period is not less than 1 inch in 24 hours.

Of the 4,316 stations whose records were searched, 1,262 were west of, and 3,054 east of, the one hundred third meridian which passes about 110 miles east of Denver, Colorado. An average of about ten records of excessive precipitation was found for each station. Since the rainfall conditions in the western part of the United States differ materially from those in the eastern part, no study was given to the precipitation occurring west of the one hundred third meridian nor were the records of precipitation prior to 1870 considered in their final study, because of the

small number of stations in existence before that date. From 1870 to 1914, a total of 2,641 periods of excessive precipitation were found Of these, 1,236 were recorded at one station only, 996 at two to five stations, and 409 at six or more stations. The data by years are given in Table IV.

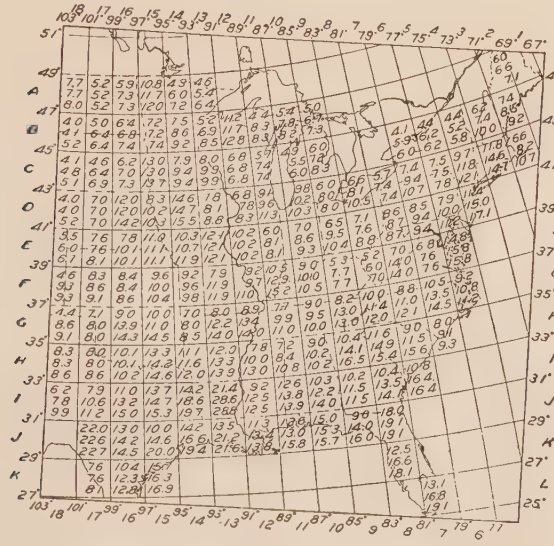
TABLE IV.[1]—TOTAL NUMBER OF RAINFALLS CAUSING EXCESSIVE PRECIPITATION RECORDED IN EASTERN UNITED STATES AT ONE STATION, AT TWO TO FIVE STATIONS, AND AT SIX OR MORE STATIONS, FROM 1871 TO 1914, INCLUSIVE

Year	Storms recorded at			Total number of storms	Year	Storms recorded at			Total number of storms
	One station	Two to five stations	Six or more stations			One station	Two to five stations	Six or more stations	
1871	20	3	0	23	1893	23	24	2	49
1872	16	13	0	29	1894	30	19	7	56
1873	24	8	0	32	1895	24	31	8	63
1874	29	13	1	43	1896	19	32	16	67
1875	25	20	1	46	1897	32	33	12	77
1876	35	17	1	53	1898	38	31	16	85
1877	30	19	1	50	1899	17	30	17	64
1878	27	16	3	46	1900	37	31	24	92
1879	32	13	2	47	1901	28	23	13	64
1880	29	23	2	54	1902	26	33	25	84
1881	29	11	3	43	1903	29	22	22	73
1882	39	11	3	53	1904	38	29	14	81
1883	33	11	1	45	1905	35	39	25	99
1884	32	16	1	49	1906	33	30	20	83
1885	22	13	2	37	1907	40	30	16	86
1886	28	12	1	41	1908	30	41	18	89
1887	19	7	3	29	1909	19	29	21	69
1888	31	14	4	49	1910	24	31	10	65
1889	26	21	3	50	1911	22	29	21	72
1890	26	20	4	50	1912	32	41	16	89
1891	22	18	8	48	1913	36	32	15	83
1892	27	28	6	61	1914	23	29	21	73
Totals						1,236	996	409	2,641

[1] From *Technical Report*, part V, Miami Conservancy District.

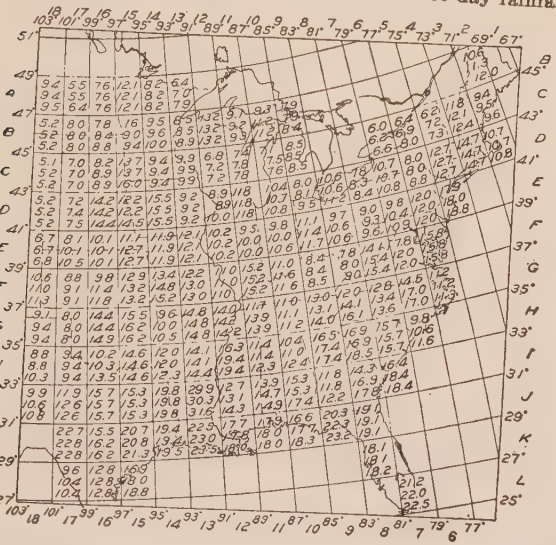
A study of the table shows: (1) that the number of storms recorded at one station was fairly uniform throughout the period; (2) that the number recorded at two to five stations increased approximately 100 per cent during the last half of the period; and (3) that the number of storms recorded at six or more stations was several times as large from 1896 to 1914 as from 1870 to 1896. These differences are due not to a progressive increase in the amount of precipitation and in the area covered, but to the

larger number of observing stations which existed during the latter period.



(D. W. Mead's "*Hydrology.*")

FIG. 7.—Maximum depths of one, two, and three day rainfalls.



(D. W. Mead's "*Hydrology.*")

FIG. 8.—Maximum depths of four, five, and six day rainfalls.

During the 20-year period from 1875 to 1894, the average number of storms recorded annually was 28 at one station, 17

at two to five stations, and 3 at six or more stations; while during the 20-year period from 1895 to 1914, the average number of storms annually was 28, 31, and 18 for one station, two to five stations, and six or more stations, respectively. Almost one-half of the total number of storms was recorded at one station

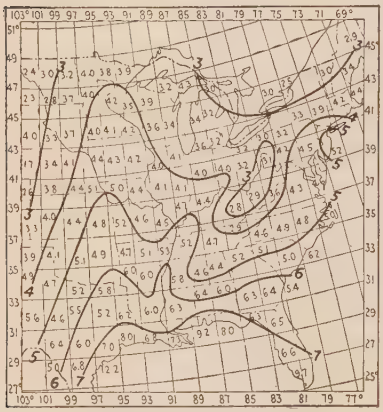


FIG. 9.—Isopluvial chart for 15-year period and 1-day rainfall.

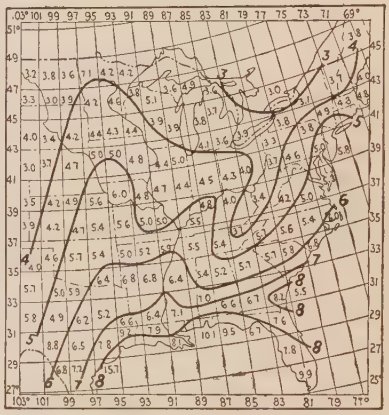


FIG. 10.—Isopluvial chart for 25-year period and 1-day rainfall.

FIG. 11.—Isopluvial chart for 50-year period and 1-day rainfall.

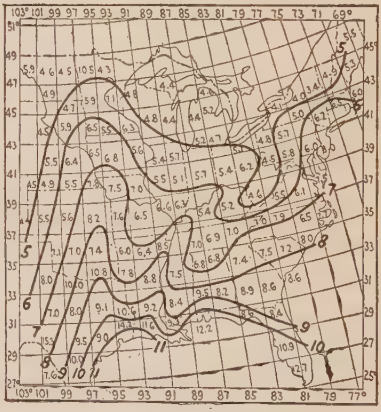


FIG. 12.—Isopluvial chart for 100-year period and 1-day rainfall.

only which would indicate that they were local storms of intense precipitation. One-day and some two-day storms are usually local, whereas those of longer duration cover large areas.

In compiling and analyzing the data for these storms, the United States was divided into 2-degree quadrangles. The

maximum precipitation ever recorded in each of these quadrangles during periods of from 1 to 6 days is shown in Figs. 7 and 8.

Isopluvial Charts.—From these data a series of charts were prepared showing by isopluvial lines the amount of precipitation

FIG. 13.—Isopluvial chart for 15-year period and 2-day rainfall.

FIG. 14.—Isopluvial chart for 25-year period and 2-day rainfall.

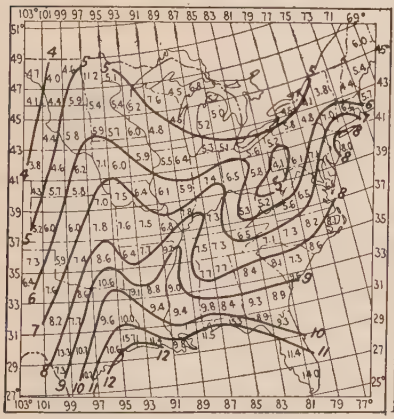


FIG. 15.—Isopluvial chart for 50-year period and 2-day rainfall.

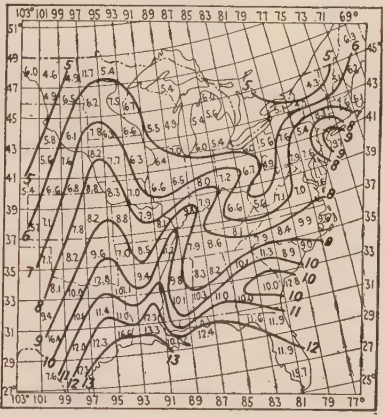


FIG. 16.—Isopluvial chart for 100-year period and 2-day rainfall.

which might be expected to occur at any place in periods of from 1 to 6 days with a frequency of once in 15, 25, 50, and 100 years. These charts are reproduced in Figs. 9 to 32.

In constructing the isopluvial lines for 15- and 25-year frequencies, all station records less than 5 years in length were omitted;

and for the 50- and 100-year frequencies, records less than 10 years in length were omitted. The isopluvial lines for frequencies of 15 and 25 years are more reliable than those for the longer frequencies, because of the larger number of observations;

FIG. 17.—Isopluvial chart for 15-year period and 3-day rainfall.

FIG. 18.—Isopluvial chart for 25-year period and 3-day rainfall.

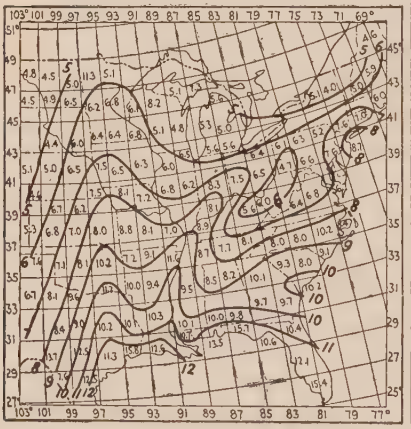


FIG. 19.—Isopluvial chart for 50-year period and 3-day rainfall.

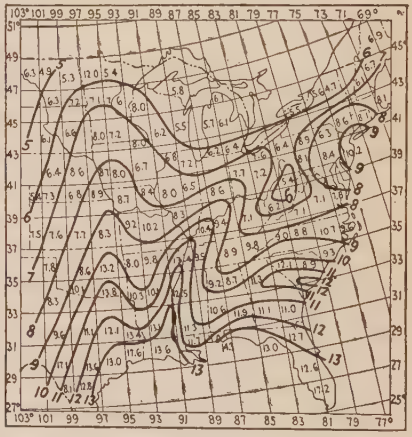


FIG. 20.—Isopluvial chart for 100-year period and 3-day rainfall.

also, all the lines are more reliable in those parts of the country where long records exist.

The effects of latitude, topography, and proximity to water areas are clearly apparent on all the charts. As latitude increases and the distance from large bodies of water increases,

the magnitude of storms decreases. The general east and west direction of the isopluvial lines is modified by the mountains and valleys. The Ozark and Appalachain mountains cause a southward deflection; and the Mississippi, Ohio, and Missouri valleys cause a northward deflection.

FIG. 21.—Isopluvial chart for 15-year period and 4-day rainfall.

FIG. 22.—Isopluvial chart for 25-year period and 4-day rainfall.

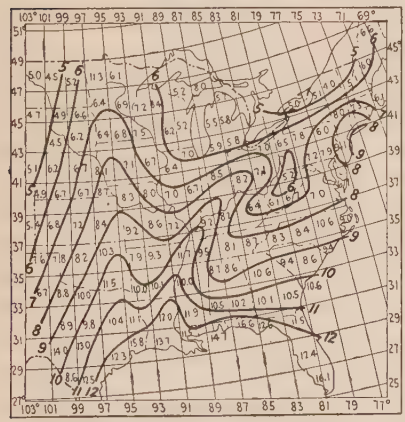


FIG. 23.—Isopluvial chart for 50-year period and 4-day rainfall.

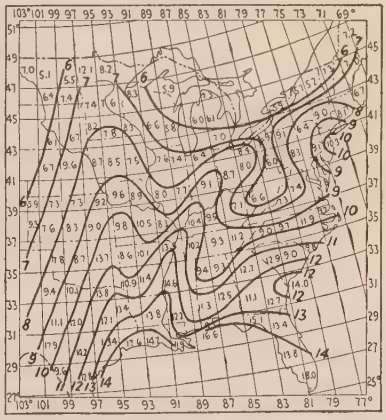


FIG. 24.—Isopluvial chart for 100-year period and 4-day rainfall.

Since the isopluvial lines are based on the maximum precipitation which has occurred at any station in each quadrangle, their use is limited to comparatively small areas. They are very valuable in supplying a basis of design for land drainage systems and for flood protection of small watersheds. For large water-

sheds, however, the area covered by excessive storms is of as much importance as the depth and duration of precipitation. For a study of the three factors of area, depth, and duration, the engineer is limited to the larger storms.

FIG. 25.—Isopluvial chart for 15-year period and 5-day rainfall.

FIG. 26.—Isopluvial chart for 25-year period and 5-day rainfall.

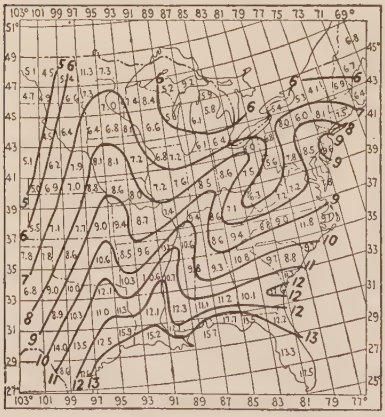


FIG. 27.—Isopluvial chart for 50-year period and 5-day rainfall.

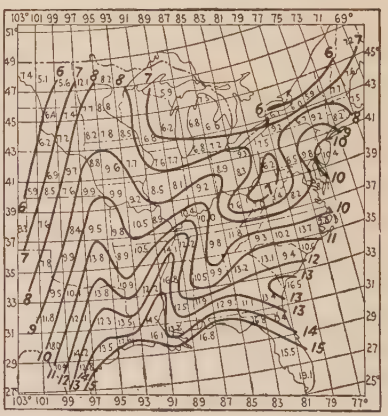


FIG. 28.—Isopluvial chart for 100-year period and 5-day rainfall.

Analysis of 160 Great Storms.—The engineers of the Miami Conservancy District made a careful study of 160 storms occurring between 1892 and 1916 which were of sufficient size to be recorded at five or more stations, and in which the precipitation for a 3-day period equaled or exceeded 6 inches. Since it was early apparent that the storms on the Gulf coast and the southern

seaboard were of much greater magnitude and intensity than those in the northern and eastern states, the 160 storms were divided into two groups—the northern and the southern—and analyzed separately. The northern boundaries of North Carolina, Tennessee, Arkansas, and Oklahoma were taken arbitrarily as

FIG. 29.—Isopluvial chart for 15-year period and 6-day rainfall.

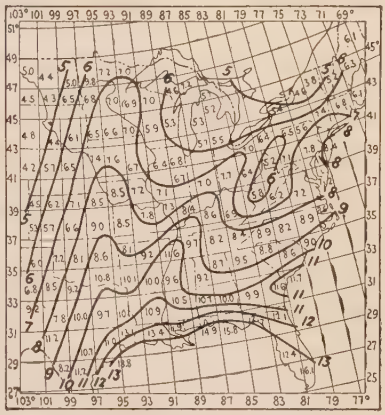


FIG. 30.—Isopluvial chart for 25-year period and 6-day rainfall.

FIG. 31.—Isopluvial chart for 50-year period and 6-day rainfall.

FIG. 32.—Isopluvial chart for 100-year period and 6-day rainfall.

the dividing line. In the northern group there were 47 storms, and in the southern group 113 storms. Each group was subdivided into four seasonal groups, according to the time of the year when the storms occurred. In dividing the year into quarters, November was taken as the first month. The number

of storms occurring during each month and each quarter, the average area covered, and the average depth of precipitation within the 6-inch isohyetals are given in Table V.

Geographical and Seasonal Distribution.—Of the 47 storms in the northern group, only six occurred during the first quarter, all of which were over the lower Mississippi and Missouri river valleys. Only two were recorded during the second quarter, one of which was the storm of Mar. 23 to 27, 1913, which caused so much damage in the Ohio Valley. During the third quarter, there were 15 storms most of which were over the Mississippi and Missouri river valleys and were characterized by excessive precipitation of short duration over comparatively small areas. The largest number of storms occurred during the fourth quarter, when 24 were recorded. These storms were about equally divided between the Atlantic coast region and the lower Mississippi and Missouri river valleys.

TABLE V.—MONTHLY DISTRIBUTION OF 160 GREAT STORMS OCCURRING IN THE 25-YEAR PERIOD FROM 1892 TO 1916

Month and quarter	Northern storms			Southern storms		
	Number	Average area,[1] square miles	Average depth, inches	Number	Average area,[1] square miles	Average depth, inches
November	1	5,420	6.8	2	15,010	7.4
December	1	24,400	7.5	9	6,170	7.7
January	4	14,940	7.0	4	13,650	7.9
First quarter	6	14,930	7.0	15	9,340	7.7
February	0	.....	...	4	10,560	7.0
March	2	19,550	6.9	9	13,300	7.9
April	0	.....	...	12	13,100	7.3
Second quarter	2	19,550	6.9	25	12,760	7.5
May	3	4,240	7.2	13	10,710	7.6
June	3	3,370	7.9	7	15,270	8.4
July	9	4,800	7.7	12	19,440	8.6
Third quarter	15	4,400	7.6	32	14,980	8.1
August	6	17,680	7.4	14	15,860	8.2
September	14	4,300	7.0	19	14,130	7.7
October	4	19,120	7.4	8	11,340	8.9
Fourth quarter	24	10,120	7.2	41	14,220	8.1
Total	47	.....	...	113		

[1] Area within 6-inch isohyetals.

Of the southern storms, 15 happened during the first quarter and with two exceptions were centered over the lower Mississippi Valley. During the second quarter, 25 storms were recorded. These were distributed fairly uniformly over the southern states with the exception of Florida, and all were far enough inland for the 6-inch isohyetals to fall within the land area. The 32 storms of the third quarter were quite similar in general characteristics to those of the second quarter, but they extended from central Texas to the Atlantic coast and were somewhat concentrated along the Gulf coast. The greatest storms occurred during this period. The storms of the fourth quarter were more numerous but were otherwise about the same as those of the third quarter. Quoting from *Technical Report*, Part V:

> . . . during the months of November to April, all storms in both groups occur in the Mississippi Valley from the Gulf of Mexico to Iowa, Illinois, Indiana, and Ohio. From May to October, they occur principally along the Atlantic and Gulf coastal regions and in the central Mississippi Valley, the greatest number of the interior storms occurring west of the Mississippi River.

In the north, excessive storms are more frequent during the months of July, August, and September; 62 per cent occurred during these 3 months. In the south, 62 per cent of the storms took place during April, May, July, August, and September.

Area Covered.—The minimum area, within the 6-inch isohyetal, covered by any northern storm was 520 square miles; and by any southern storm, 1,100 square miles. The maximums for the two groups were 87,200 and 76,800 square miles, respectively. The data in Table VI show that the southern storms covered much larger areas than the northern. Of the northern storms, 55.4 per cent covered less than 5,000 square miles, 25.5 per cent between 5,000 and 10,000, 10.7 per cent between 10,000 and 20,000, 4.2 per cent between 20,000 and 40,000, and 4.2 per cent over 40,000 square miles. For the southern storms, the percentages for the same areas were 19.4, 31.0, 31.0, 15.0, and 3.6, respectively.

Depth of Precipitation.—The average depths of precipitation over the areas within the 6-inch isohyetals for the four quarters were 7.0, 6.9, 7.6, and 7.2 inches, respectively, for the northern storms, and 7.7, 7.5, 8.1, and 8.1 inches, respectively, for the southern storms, showing an increase of from 10 to 15 per cent in the intensity of the latter storms.

TABLE VI.—AREA OF STORMS WITHIN THE 6-INCH ISOHYETAL

Area of storm, square miles	Northern storms		Southern storms	
	Number	Per cent	Number	Per cent
500– 1,000	1[1]	2.1	0	0
1,000– 2,000	6	12.8	1[2]	0.9
2,000– 3,000	6	12.8	5	4.4
3,000– 4,000	6	12.8	4	3.5
4,000– 5,000	7	14.9	12	10.6
5,000– 7,500	7	14.9	20	17.7
7,500–10,000	5	10.6	15	13.3
10,000–15,000	2	4.3	26	22.9
15,000–20,000	3	6.4	9	8.1
20,000–30,000	1	2.1	12	10.6
30,000–40,000	1	2.1	5	4.4
40,000–70,000	1	2.1	3	2.7
70,000–up	1[1]	2.1	1[2]	0.9
Total	47	100.0	113	100.0

[1] Maximum, 87,200; minimum, 520.
[2] Maximum, 76,800; minimum, 1,100.

Depth-area-time Relation.—Further study was made of the 33 largest storms to determine a relation between the depth and duration of the precipitation and the area covered by the storm. Figures 33 and 34 show this relation for the two groups of storms. The full-line curves give the areas covered by precipitations of different depths for the maximum 1- to 5-day storms. The dotted-line curves give the same data for average conditions, and were obtained by taking the averages of all the large storms. The following conclusions can be drawn from these curves:

1. More than half of the total precipitation during a storm period occurs in some one day of that period.

2. In general, there is little difference between the largest storm and the second or third largest.

3. Southern storms are of much larger size and of greater intensity than northern storms.

4. The depth of precipitation decreases at a much greater rate than the area increases.

5. The increase in depth of precipitation with an increase in time is quite marked up through the 3-day period, beyond which time there is little, if any, increase. In fact, in the northern group, the greatest 4- and 5-day storms are less than the greatest 3-day storm.

6. Although not indicated by the curves, the maximum 1-day precipitation rarely occurred on the first or last day of the storm period.

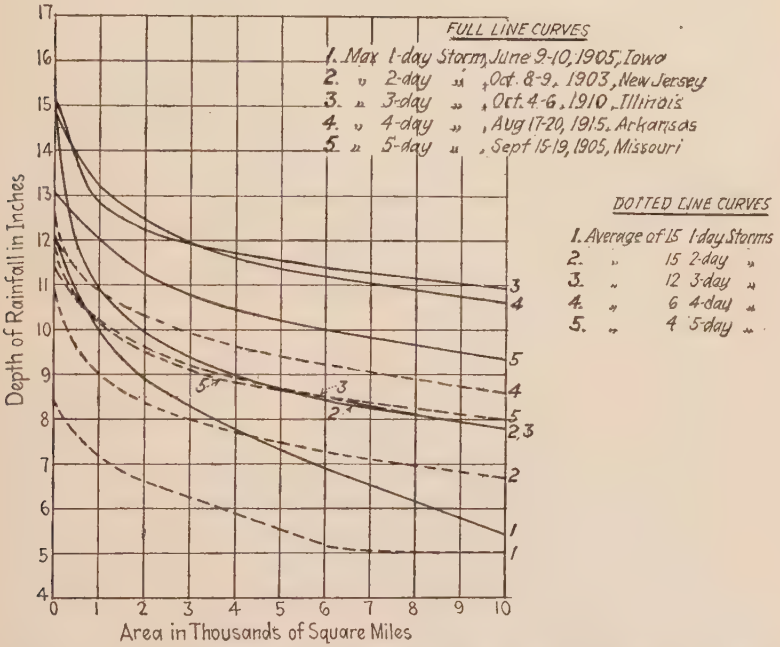

FULL LINE CURVES
1. Max 1-day Storm, June 9-10, 1905, Iowa
2. " 2-day " , Oct. 8-9, 1903, New Jersey
3. " 3-day " , Oct. 4-6, 1910, Illinois
4. " 4-day " , Aug 17-20, 1915, Arkansas
5. " 5-day " , Sept 15-19, 1905, Missouri
DOTTED LINE CURVES
1. Average of 15 1-day Storms
2. " 15 2-day "
3. " 12 3-day "
4. " 6 4-day "
5. " 4 5-day "
Depth of Rainfall in Inches
Area in Thousands of Square Miles

FIG. 33.—Curves showing the relation between area covered by storm and depth of rainfall for 1- to 5-day storms over northern states.

Intense Precipitation.—Intense precipitation as here considered refers to those heavy downpours in which several inches of water are precipitated within a period of 1 or 2 hours or even less. Such precipitations are the basis of the design of storm sewers, spillways, flood gates in dams, and drainage channels under certain conditions.

Intense precipitations cover small areas and, since our observing stations are widely scattered, undoubtedly a large number occur without any record being obtained of them; also, since only about 200 observing stations are equipped with automatic

rain gages, most intense precipitations are not recorded separately, but are included in the total rainfall for 24-hour periods.

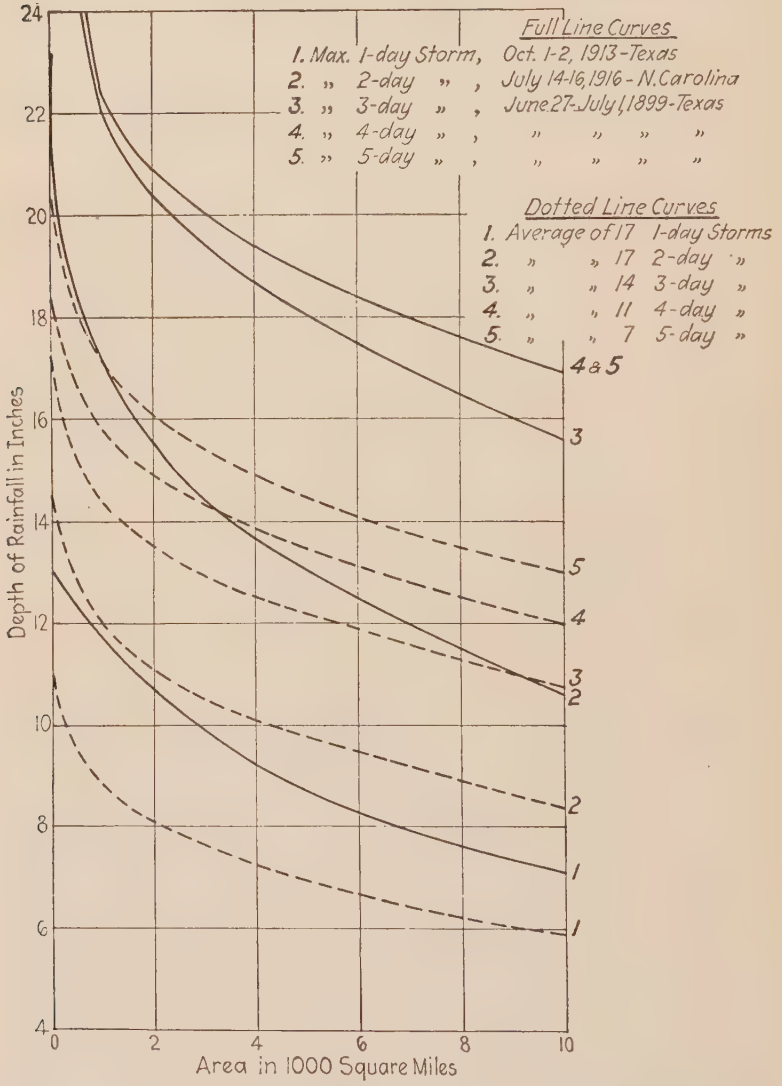
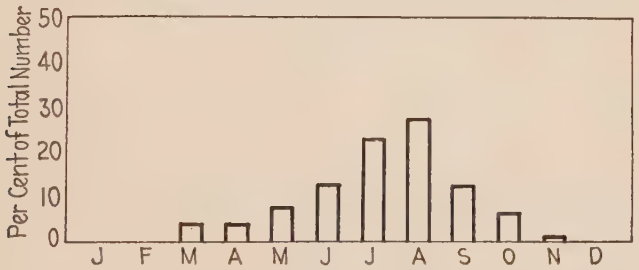


FIG. 34.—Curves showing the relation between area covered by storm and depth of rainfall for 1- to 5-day storms over southern states.

Hence, our records of such precipitation are comparatively meager.

Geographic Distribution.—In general it may be said that the greater the annual rainfall, the more intense and frequent are the intense precipitations which occur. This is not a fast rule, however, and there are records of such precipitations in the North and West as great as those which occur in the southern states. Meyer in his "Elements of Hydrology," Table 13, gives the most exceptional rates of precipitation during 100 intense rainfalls. Of these, 39 occurred in the South, 17 in the Northeast, 32 in the Middle West, and 12 in the western states east of the Rocky Mountains. These data show that up to a period of 60 minutes, there is little difference in the intensity of precipitation in different parts of the United States; but for longer periods, the southern storms are somewhat more intense than the others.

Time of Occurrence.—The monthly distribution of the 100 intense storms referred to above is shown in Fig. 35. None occurred during the months of December, January, and February,



(Compiled from Table 13, Meyer's "*Elements of Hydrology.*")

Fig. 35.—Monthly distribution of 100 intense rainstorms from 1896 to 1914.

and only one in November. During July and August 51 per cent occurred, and 77 per cent during the months of June, July, August, and September. These are the months when flooding of land is the most injurious from an agricultural viewpoint.

Frequency of Occurrence.—As the result of the study of 1,962 storms occurring between 1896 and 1914, Meyer derived formulas, which best fitted the observed data, for the amount and rate of precipitation for frequencies of 1, 5, 10, 25, 50, and 100 years. The rates of precipitation which would probably be exceeded once in 1, 2, 5, and 10 years were selected from the 19-year records of individual observation stations. The stations were then divided into five groups on the basis of similar rates of precipitation. The records of the stations in each group were

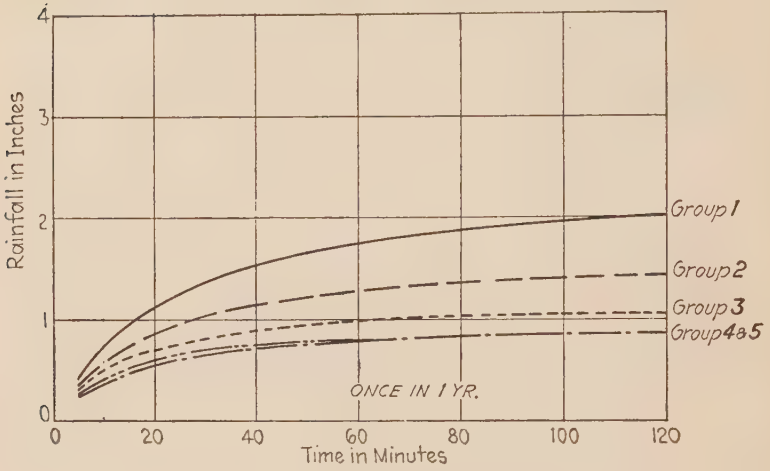


FIG. 36.—Frequency curves of intense precipitation as determined by Meyer's formulas.

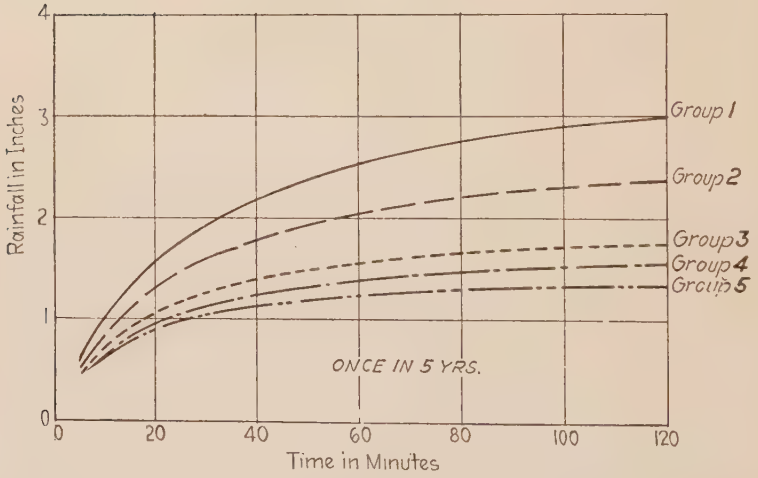


FIG. 37.—Frequency curves of intense precipitation as determined by Meyer's formulas.

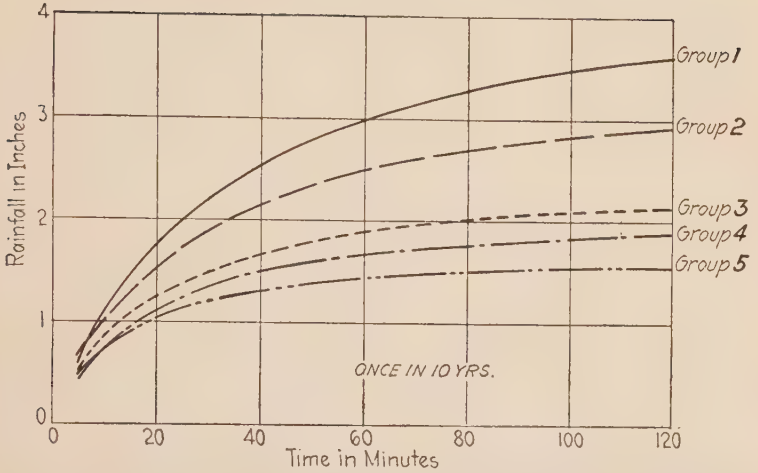


FIG. 38.—Frequency curves of intense precipitation as determined by Meyer's formulas.

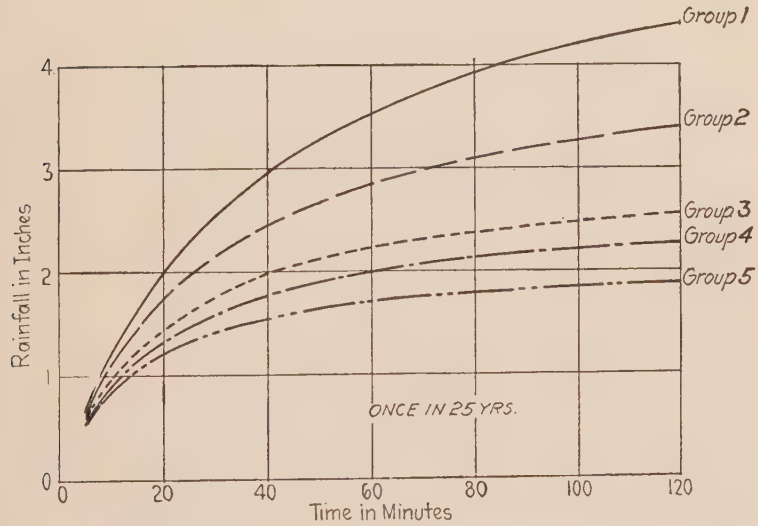


FIG. 39.—Frequency curves of intense precipitation as determined by Meyer's formulas.

then combined and considered as if all the precipitations had occurred at one station during a period equal to the sum of the lengths of the individual records. From this combined record for each group, formulas were developed for the rates of precipitation which would probably be exceeded in 1, 2, 5, 10, 25, 50, and 100 years. The observation stations upon which these formulas are based are as follows: *Group* 1—Galveston, New Orleans, and Jacksonville; *Group* 2—New York, Philadelphia, Washington, Norfolk, Raleigh, Savannah, Atlanta, Little Rock, Fort Worth, Abilene, Bentonville, St. Louis, Kansas City, Lincoln, and Des Moines; *Group* 3—Boston, Albany, Pittsburgh, Elkins, Asheville, Knoxville, Memphis, Cairo, Indianapolis, Cincinnati, Cleveland, Detroit, Grand Haven, Chicago, Madison, St. Paul, Moorhead, Yorkton, and Dodge City; *Group* 4—Duluth, Escanaba, Rochester, and Buffalo; and *Group* 5—Denver and Bismark. The curves in Figs. 36 to 39 were constructed from data in Table 15, Meyer's "Elements of Hydrology," and show the relation between time and amount of rainfall for frequencies of 1, 5, 10, and 25 years.

There is no regularity in the occurrence of intense precipitation. When it is stated that a storm of given intensity will occur with a frequency of once in 10 years, it is understood that when a long period of time is considered such a storm is likely to occur on an average of once in 10 years. As an example of intense rainfall occurring over the same watershed at short intervals, the series of intense rainfalls over the Lackawanna Valley of Pennsylvania during the summer of 1922 may be cited. These storms occurred on June 3, 11, 17, and 20 ,and July 1, 2, and 28. The rainfall of June 3 was centered to the south of Carbondale. No record of the amount was obtained, but the maximum rate of runoff from Wilson Creek, with a drainage area of 3.4 square miles, reached a maximum of about 300 cubic feet per second per square mile; and that from Fall Brook, with a drainage area of 11.4 square miles, was about 146 cubic feet per second per square mile. No record of the rainfall of June 11 was obtained. This storm was severe but of short duration. On June 17, the rainfall on the Fall Brook watershed was 4.3 inches in 4.3 hours. On June 20, 3.5 inches of rain fell in 2 hours. The storm of July 1 was of short duration but of great intensity; no record was obtained. On July 2, the whole upper drainage area of the Lackawanna River experienced an unusually intense rainfall. Over the Fall

Brook watershed, 2.75 inches fell in 75 minutes. The storm of July 28 was centered over Scranton. The precipitation was 1.91 inches in 1 hour at the United States Weather Bureau Station. This intensity is said to have been equaled only once in the 22 years since the station was established.[1]

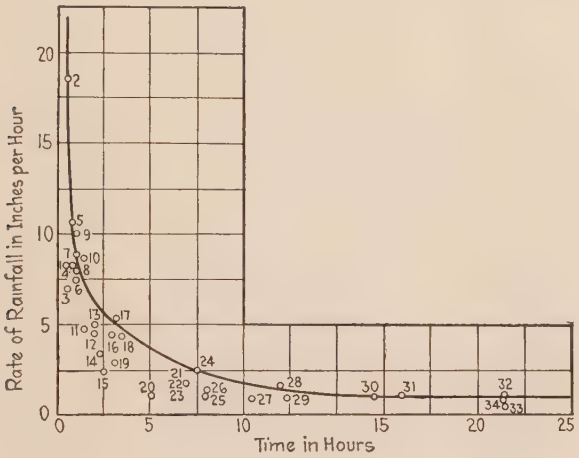


FIG. 40.—Unusually intense rates of rainfall.

Table VII gives a list of 34 unusually intense rainfalls which have occurred in periods of 24 hours or less. These rates of precipitation are shown plotted in Fig. 40 as ordinates with their durations as abscissas, and a curve drawn through the higher values to show the maximum intensities which may possibly occur in the different periods of time.

[1] *Eng. News-Record*, vol. 90, No. 18, p. 798.

TABLE VII.[1]—UNUSUALLY INTENSE RAINFALLS FOR DURATIONS OF 24 HOURS OR LESS

Reference number	Location	Date	Amount, Inches	Duration		Rate in inches per hour
				Hours	Minutes	
1	Biscayne, Fla.	Mar. 28, 1874	4.10	0	30	8.20
2	Guinea, Va.	Aug. 24, 1906	9.25[2]	0	30	18.50
3	Logansport, Md.	July 7, 1879	3.50	0	30	7.00
4	Newton Square, Pa.	Aug. 5, 1843	5.5[2]	0	40	8.25
5	Fort Mohave, Ariz.	Aug. 28, 1898	8.00[2]	0	45	10.67
6	Tridelphia, W. Va.	July 19, 1888	6.90	0	55	7.53
7	Palmetto, Nev.	Aug. , 1890	8.80	1	00	8.80
8	Cincinnati, Ohio.	June 3, 1921	8.0[2]	1	00	8.00
9	Catskill, N. Y.	July 26, 1819	10.0[2]	1	00	10.00
10	Campo, Cal.	Aug. 12, 1891	11.5	1	20	8.63
11	Cambridge, Ohio.	July 16, 1914	7.09[3]	1	30	4.73
12	Taborton, N. Y.	Aug. 10, 1920	8.95[2]	2	00	4.47
13	Brandywine Hundred, Del.	Aug. 5, 1843	10.00[2]	2	00	5.00
14	Roney's Point, W. Va.	July 19, 1888	8.0[2]	2	20	3.43
15	Fort Meade, S. Dak.	June 13, 1907	6.01	2	30	2.40
16	Catskill, N. Y.	July 26, 1819	13.00[2]	3	00	4.33
17	Concord, Pa.	Aug. 5, 1843	16.0[2]	3	00	5.33
18	Newton Twp., Pa.	Aug. 5, 1843	13.0[2]	3	00	4.33
19	Alton, Ill.	July 13, 1912	9.22[2]	3	10	2.91
20	Albuquerque, N. Mex.	Oct. 9, 1865	5.20	5	00	1.04
21	Quanah, Tex.	June 4, 1891	12.25	7	00	1.75
22	Boggs Flats, Colo.	June 3, 1921	12.0[2]	7	00	1.71
23	York, Pa.	June 25, 1884	12.0[2]	7	00	1.71
24	Catskill, N. Y.	July 26, 1819	18.0[2]	7	30	2.40
25	Flatbush, N. Y.	Aug. 22, 1843	9.0	8	00	1.12
26	Griffin, Ga.	Aug. 8, 1883	10.38	8	00	1.30
27	White Plains, N. Y.	Oct. 9, 1877	9.70	10	30	0.92
28	Taylor, Tex.	Sept. 9, 1921	19.5	12	00	1.63
29	Larrabee, Iowa.	June 23, 1891	12.95	12	24	1.04
30	St. George, Ga.	Aug. 28, 1911	18.00	17	00	1.06
31	Montell, Tex.	June 29, 1913	20.6	18	30	1.11
32	Hearne, Tex.	June 28, 1899	24.0	24	00	1.00
33	Beaulieu, Minn.	July 20, 1909	10.75	24	00	0.45
34	Altapass, N. C.	July 15, 1916	22.2	24	00	0.92

[1] IVAN E. HOUK, "Hydraulic Design of Bridge Waterways," *Eng. News-Record*, vol. 88, No. 26, p. 1072.

[2] Believed reliable although not measured in a standard rain gage.

[3] Greater part of the rainfall fell in less than 30 minutes.

Formulas for Intense Precipitation.—Other formulas for intense precipitation and the conditions to which they apply are given in Table VIII. In these formulas, i is the rate of precipitation in inches per hour, and t is the duration of the storm in minutes.

TABLE VIII.—FORMULAS FOR INTENSE PRECIPITATION

Author	Formula	Remarks
Metcalf and Eddy...	$i = \frac{15.5}{t^{0.5}}$	Boston, Mass.
Metcalf and Eddy...	$i = \frac{14}{t^{0.5}}$	Louisville, Ky.
Metcalf and Eddy...	$i = \frac{19}{t^{0.5}}$	New Orleans, La.
Metcalf and Eddy...	$i = \frac{84}{t + 4}$	Denver, Colo.
Talbot, A. N.....	$i = \frac{360}{t + 30}$	Maximum, east of Rocky Mountains
Talbot, A. N.....	$i = \frac{105}{t + 15}$	Ordinary, east of Rocky Mountains
Kuichling, Emil.....	$i = \frac{120}{t + 20}$	Boston, Mass.
Horner.....	$i = \frac{56}{(t + 5)^{0.85}}$	Basis of design, St. Louis, Mo.
Hill.....	$i = \frac{120}{t + 15}$	Chicago, Ill.
Bureau of Survey....	$i = \frac{9}{t^{0.5}}$	Ordinary, Philadelphia, Pa.
Sherman, C. W.....	$i = \frac{25.12}{t^{0.687}}$	Ordinary, Boston, Mass.
Sherman, C. W.....	$i = \frac{38.64}{t^{0.687}}$	Maximum, Boston, Mass.
Gregory, C. E.....	$i = \frac{12}{t^{0.5}}$	Ordinary, Boston, Mass.

CHAPTER III

FLOOD RUN-OFF

Run-off or stream flow is that portion of the precipitation which finds its way into natural or artificial channels either as surface flow during a storm period or as subsurface flow after the storm period has passed. The surface or flood run-off is that with which the flood control and drainage engineer has to deal, and is the basis for the design of all his structures.

Run-off is best determined by measuring the volume of water which is discharged through the stream which drains the watershed. The discharge of a stream varies greatly during the year and from year to year, and the probable maximum discharge can be satisfactorily estimated only after discharge measurements have been made for a number of years. The longer the record of such measurements, the more accurate will be the conclusions drawn from it. In flood control and drainage work, it is not the ordinary or average flow which governs the design, but the exceptional flow which will occur with a frequency of from once in 10 years to once in 50 or 100 years. Records of stream flow are available for most of the large streams of the United States, but for few of the numerous smaller streams; and the records of the latter generally cover too short a period to be of much value when considered independently. When the need arises for stream-flow data in connection with engineering projects, there is not time for an extended series of measurements to be made, since the project cannot be delayed; and the engineer is forced to estimate the probable maximum run-off from a study of the factors which cause run-off and by a comparison with more or less similar streams for which data are available, supplemented by the few measurements which can be made on the stream itself in the short time allowed for such investigations.

Units of Run-off Measurement.—Total run-off is expressed in inches of depth over the entire watershed in the course of a year. In this way the run-off can be easily compared with the annual precipitation which is expressed in the same units. In dealing with flood run-off, however, it is the maximum rate of

run-off which is of importance rather than the total amount of run-off in a year, a month, or even a day. So long as the stream draining the watershed does not overflow its banks, there will be no flood problem. The total run-off during a storm period may be quite large, but if it is uniformly distributed throughout the period—provided the period is not too short—the water will be carried away in the channel provided for it at less than bankful stage.

Flood run-off is expressed in three ways: (1) as the volume of discharge of the stream at the point under consideration in one second of time; (2) as the volume of discharge in a second of time for each square mile of tributary area; and (3) as the depth in inches over the tributary area which is carried off by the stream in a period of 24 hours. For example, if the watershed area contributing to the flood flow is 3,000 square miles and the discharge of the stream is 50,000 cubic feet per second, the run-off is said to be either 50,000 second-feet, 17 second-feet per square mile, or 0.63 inch in 24 hours. The last expression is generally used for small areas such as might be contained in a drainage district, and is called the drainage modulus.

Run-off Factors.—Although the factors which affect run-off are many and varied, they can all be grouped under two heads: (1) precipitation; and (2) watershed characteristics. Of these, the former is of greater importance, for without precipitation there can be no run-off. Other factors remaining constant, the greater the precipitation the greater the run-off, although not necessarily in direct proportion. The occurrence, distribution, and frequency of precipitation have already been discussed in Chapter II, so in this chapter it will be necessary to consider only the effect of precipitation on run-off.

The effect of precipitations of varying length and intensity is much easier to evaluate than is the effect of watershed characteristics, because the latter are numerous and occur in so many combinations; hence it will be necessary to study these characteristics before proceeding further.

WATERSHED CHARACTERISTICS

Location of Watershed.—The geographic location of a watershed has an important bearing on the amount and frequency of flood run-off. As shown in Figs. 33 and 34, southern watersheds are subject to more frequent and more excessive storms than are

northern ones, and other factors being similar, the flood run-off from them will be greater. The same is true of those watersheds lying in the more frequented storm paths. Also a watershed whose length is several times its width will produce a greater flood run-off if its major axis lies parallel to storm paths than if at right angles to them, since in the former instance, precipitation will occur over the entire watershed, while in the latter, over only a portion of it; furthermore, in the former instance, if the watershed drains in the same direction as that taken by storms, it will experience more severe floods than if it drained in the opposite direction, due to the concentration of the run-off. Thus streams flowing east are more likely to cause floods than those flowing west, other conditions being similar.

In the chapter on precipitation, the effect of temperature on the amount of precipitation was pointed out. In that discussion the precipitation considered was in the form of rain. Temperature has, however, another bearing on run-off aside from the amount of precipitation. In northern latitudes the precipitation during the winter months is largely in the form of snow which accumulates on the ground, often to a considerable depth. At the same time the rivers are frozen over, and stream flow is thereby retarded. If the rise of temperature in the spring is gradual, with the temperature above freezing during the day, followed by freezing temperatures at night, the snow and ice will melt slowly and intermittently and the water will be carried away by the streams without flooding. On the other hand, a sudden rise of temperature with night temperatures above freezing will result in a rapid melting of the snow and ice, and the water running over the frozen surface of the ground will quickly reach the tributary channels and flow thence into the main stream, more or less choked with floating ice, causing a greater or less flood

When the rise in temperature is accompanied by rain, the flood run-off is greatly increased, and flood conditions are almost sure to follow. This is about the most unfavorable combination of conditions which could happen and is responsible for most of the disastrous spring floods which have occurred in the North.

For a large watershed whose major axis lies north and south, a gradual increase in temperature will melt the snow at the southern end of the watershed first, allowing it to run off before the snow at the northern end is melted, provided the flow is to the

south; but, if the watershed drains to the north, the conditions are reversed and the water from the melting snow at the upper end of the watershed will be augmented as it flows downstream by the melting snow along the way, and the flood crest will increase in height as it moves downstream. Since the ice in the stream breaks up at the upper end first, it will be piled up on the unbroken ice below, forming an ice jam which will retard the flow of the water, raise the flood height above, and increase the likelihood of serious flooding. Since most of the large streams in the United States, which have their major axes approximately north and south, drain to the south, or are situated far enough south to be free from any considerable amount of snow, this condition is of infrequent occurrence in this country. The Red River of the North is the most prominent stream subject to floods from this cause.

If the major axis of a wide watershed is approximately east and west, both ends of the watershed will experience the same temperature at the same time, and the snow will melt at the same rate at both ends whether the rise in temperature be gradual or sudden. Obviously, the higher the temperature, the greater will be the rate of melting and the rate of run-off. For a gradual rise in temperature, however, the snow on the southern portion of such a watershed will melt first and be carried away before that on the northern portion melts. The best illustration of a watershed of this kind is that of the Ohio River.

Thus it is seen that the location of a watershed, with respect to latitude and the direction of drainage, has a great deal to do with the flood discharge which it may produce.

Size of Watershed.—The size of a watershed affects the rate of run-off and the manner of its occurrence. On large watersheds floods reach their crest slowly, remain at flood stage for days or weeks, and subside slowly; while on small watersheds the flood waters collect quickly, remain at flood stage for only a short period, and subside quickly. When large streams overflow, the damage which results is more serious than in the case of small ones because of the greater extent of their flood plains and of the greater length of flood period; but the suddenness of floods on small watersheds, especially those occurring at night, frequently cause a heavy loss of life and property.

Intense precipitation occurs over small areas only, and is the kind which causes floods on small watersheds. Such a precipita-

tion over a large watershed will have little effect on the flow of the main stream. It is the steady rainfall lasting several days and covering the entire watershed which causes floods on large watersheds. Small watersheds may consist entirely of steep slopes and impervious soil, and practically all the precipitation will run off over the surface and quickly reach the stream, causing a high concentration of flow; while the topography and soil of large watersheds are more varied and the run-off is less concentrated. The rate of run-off from small watersheds may be quite high, for example, the June 1903 flood on Willow Creek, near Heppner, Oregon, with a watershed of 20 square miles, produced a flood run-off of 1,800 cubic feet per second per square mile. In contrast to this, the flood run-off of the 1913 flood on the Mississippi River with a drainage area of 1,240,000 square miles was only 2 second-feet per square mile; and that of the 1904 flood on the Illinois River with a drainage area of 27,900 square miles was 4.48 second-feet per square mile.

Shape of Watershed.—A watershed which is long and narrow is likely to have a lower rate of run-off than one which is more compact. In the former instance, the tributaries will be comparatively short and numerous and their several drainage areas will be small and the run-off from them will reach the main stream quickly and be carried away in a series of small flood crests, provided the duration of storms is short. As the length of the storm period increases, the discharge from the tributaries will combine until finally the discharge from the upper end of the watershed will reach the lowest tributary while it is still at flood stage, and the condition of maximum run-off attained. Such a watershed is not likely to be entirely covered by intense precipitation and, if the major axis is at right angles to storm paths, the entire watershed may not be covered by more general precipitations.

On the other hand, in a more compact watershed the tributaries will be longer and fewer in number and will drain larger areas, and their run-off will reach the main water course at nearly the same time, causing concentration of flow for storms of shorter duration than in the former case. A greater portion of such a watershed will be covered by intense precipitation as well as by general precipitation.

Topography of Watershed.—Naturally the slope of the surface of the ground has an important effect on flood run-off. The steeper the slopes, the greater will be the percentage of the rain-

fall which will run off over the surface, and the smaller the percentage which will be absorbed by the earth or held in depressions on the surface. Also the rate of run-off will be greater, and the time required for the run-off from the border of the watershed to reach the collecting channels will be shorter. Hence, watersheds with rolling or steep slopes will produce more frequent and greater floods than those which are more level.

A small watershed might be composed entirely of steep slopes and would yield a high rate of run-off; but it is seldom that a large watershed is entirely of this character, and the high rates of run-off from the steeper portions are counteracted more or less by the lower rates of run-off from the flatter portions.

Geology of Watershed.—The nature of the soil has a marked effect upon the quantity and intensity of run-off. If the soil is a sand, gravel, loam, or peat, a large part of the precipitation will be absorbed, leaving little for surface run-off; whereas a rocky soil will shed practically all of the water which falls upon it, and a clay soil will absorb the water so slowly that, if the surface be inclined, most of the precipitation will run off over the surface.

Vegetation.—The effect of vegetation is to retard the flow of water over the surface, to increase the percolation opportunity by opening up the soil and by retarding the flow, and to reduce the quantity and intensity of flood run-off. This is especially true for storms of short duration. After a certain amount of percolation and surface storage has been supplied, however, the surface run-off from such areas will be almost as much as though no vegetation existed, and for storms of long duration the vegetal covering has little effect.

Vegetation is also beneficial in that it prevents erosion and the formation of gullies on slopes. Water collecting in such gullies will reach the steams in a shorter time than if diffused over the surface. The material eroded from bare ground is carried to the streams where some of it is deposited in the channel, thus interfering with stream flow.

As to the effect of forests on flood run-off, there is a wide difference of opinion. The view held by some—mainly those interested in reforestation—is that deforestation on certain watersheds has resulted in increased run-off and more frequent floods. Others maintain that there has been no noticeable effect on stream flow either by deforestation or reforestation. Among the

latter are Prof. D. W. Mead[1] who, after making an exhaustive study of the effect of extensive deforestation in Wisconsin, concluded that the cutting of forests in that state had had no material effect on either the high- or the low-water flow of Wisconsin streams or on the frequency of floods. Colonel Chittenden,[2] in a notable paper presented to the American Society of Civil Engineers states that under certain conditions forests tend to increase rather than to diminish the magnitude of flood flows. He concludes in part as follows:

1. The bed of humus and débris that develops under forest cover retains precipitation during the summer season, or moderately dry periods at any time of the year, more effectively than do the soil and crops of deforested areas similarly situated. It acts as a reservoir moderating the run-off from showers and mitigating the severity of freshets, and promotes uniformity of flow at such periods.

2. The above action fails altogether in periods of prolonged and heavy precipitation, which alone produce great general floods. At such times the forest bed becomes thoroughly saturated, and water falling upon it flows off as readily as from the bare soil. Moreover, the forest storage, not being under control, flows out in swollen streams, and may, and often does, bring the accumulated waters of a series of storms in one part of the watershed upon those of another which may occur several days later: so that, not only does the forest at such times exert no restraining effect upon floods, but, by virtue of its uncontrolled reservoir action, may actually intensify them.

3. The effect of forests upon the run-off resulting from snow-melting is to concentrate it into brief periods and thereby increase the severity of freshets. This results (*a*) from the prevention of the formation of drifts, and (*b*) from the prevention of snow-melting by sun action in the spring, and the retention of the snow blanket until the arrival of hot weather.

After the great 1910 flood on the Seine River in France, the French engineers investigated the effect which reforestation of some 395,000 acres of land in the Seine Valley would have on a flood of similar magnitude. The conclusion was that the total volume of flood discharge would be decreased about 3 per cent at a cost of $35,600 per million cubic feet of water stored and

[1] "The Flow of Streams and the Factors Which Modify It, with Special Reference to Wisconsin Conditions," *Bull.* 425, University of Wisconsin.

[2] Chittenden, Col. H. M., Corps of Engineers, United States Army, "Forests and Reservoirs in Their Relation to Stream Flow, with Particular Reference to Navigable Rivers," *Trans. Amer. Soc. Civ. Eng.*, vol. 62, pp. 245–546, 1909.

transpired by the forest cover. The engineers recommended that:

> The government should not take up the reforestation of the Seine Basin directly; especially as they considered that the action of heavy forest cover in retarding run-off disappears almost entirely in the great rainfalls that cause the Seine floods; and also considering the rich agricultural lands that would be taken up for forests.[1]

The Director of the French Forestry Service in his report on the 1910 flood stated:

> In the exceptional circumstances which always produce the floods of the Seine, the action of forests, whether evergreen or deciduous, becomes for the moment negligible in its effect on the volume of water carried by the river.[2]

Surface Storage.—Lakes, ponds, and swamps tend to regulate stream flow, that is, to reduce the flood and to increase the low-water flow, thus making the flow of the stream more uniform. This is due to the fact that the outlets of such water areas are restricted and the rate of outflow from these areas is much less than the rate of inflow, causing the flood waters to be held in storage until such time as the restricted outlets can discharge them. Hence the flood flow from watersheds containing lake and swamp areas is much less than from those embracing no such areas.

Artificial Drainage.—When shallow ponds and swamps are drained by the construction of drainage ditches, the restricted outlets mentioned above as existing in a state of nature are opened up so that the possible rate of outflow from these areas is as large as the rate of inflow, thus increasing the flood flow and decreasing the low-water flow. The maximum run-off will occur early in the spring when the ground is frozen or saturated, or during extended periods of precipitation in the summer when the ground is again saturated. The purpose of drainage is, however, to lower the water table, thus removing the gravitational water in the soil, and, since swamp soils are usually loose and porous, providing a large storage reservoir in the soil itself, so that even an intense rainfall of short duration will result in a low surface

[1] GRANT, KENNETH C., "Discussion on Floods and Flood Prevention," *Trans. Amer. Soc. Civ. Eng.*, vol. 81, p. 1247, 1917.

[2] *Report* of First Commission on Flood of 1910 in the Seine, p. 515.

run-off. While the frequency of floods is thus decreased, the flood run-off from storms of long duration is increased by drainage.

Drainage of the upland level areas has, in general, the same effect. Here the soil is not so open, although cultivation and drainage tend to make it more so, and the surface run-off will be greater than in the case of the more open swamp land soil. Also when the small laterals of the drainage system are open ditches, the flood run-off will be greater than where large tiles are used for the laterals.

In general it may be stated that up to the point of soil saturation the effect of artificial drainage is to decrease flood flows, while beyond this point it is to increase it.

Cultivation.—The flood run-off from cultivated areas is less than from those which are uncultivated. Plowing breaks up the topsoil and makes it capable of absorbing more water, and that more quickly, than in the natural state; and if the furrows run parallel to the contours, as they should, a large amount of water will be stored in them before any appreciable surface flow occurs. Therefore, a watershed with a large portion of its area under cultivation will yield comparatively little surface run-off, especially during the growing season, except during extended periods of precipitation or during exceptionally intense precipitation.

Dams, Levees, and Bridges.—Dams constructed across streams back up the water and decrease the slope of the water surface, the velocity of flow, and the channel cross-section at the dam site, resulting in greater flood heights above the dam for a given run-off. The amount by which the discharge below the dam is decreased depends upon the height of dam and the depth of water over the crest of the dam. If the head of water on the crest of the dam be increased sufficiently, a stage will be reached at which the discharge below the dam will be the same as it was at that stage before the dam was constructed. Discharge measurements made before and after the construction of a 4.5-foot dam across the Susquehanna River[1] at Harrisburg, Pennsylvania, showed that when the mean depth of flow was 297 per cent of the height of the dam, the discharge was 97 per cent of that for the same depth before the dam was constructed.

The construction of levees for the reclamation of the bottom lands will also result in greater flood heights for the same discharge, and unless all the bottom land is protected by levees,

[1] *Eng. News-Record*, vol. 89, No. 18, p. 738.

the flooding of the unprotected areas will be much more frequent as the result of such leveeing. Streams have carved out their channels and flood plains to provide for their flood flows, and any encroachment by man in the way of levees, bridges, or buildings is naturally followed by more frequent flooding and greater flood heights, opinions to the contrary notwithstanding. This is becoming one of the most serious causes of floods.

Relative Effects of Watershed Characteristics.—Surface run-off is the resultant of all the characteristics of a watershed, some of which tend to hasten and others to retard the flow of the water, and it is impossible to say just what part each contributes to the general result, or to rank them in the order of their importance. The location, size, and shape of a watershed are different from the other characteristics in that they affect the run-off through the precipitation and are more easily studied and their effect evaluated.

The location of a watershed determines the nature of the precipitation, that is, whether it is all in the form of rain or partly in the form of snow and ice. It also governs the amount and manner of precipitation and the frequency of its occurrence. These are factors which can be determined by investigation.

The size of a watershed is probably the most important of all the characteristics, since upon it depends largely the amount of precipitation. Over small areas, a precipitation of high and fairly uniform intensity can occur; but as the area becomes larger the intensity becomes less uniform and the average precipitation over the watershed for a given storm becomes less. So important is the size of watershed considered that most run-off formulas contain it as the only variable, the other characteristics being either ignored or else represented by constants. The size of watershed is known in any specific case and its effect upon the total precipitation can be given proper consideration.

The shape of a watershed also has a bearing on the amount of precipitation and is considered at the same time as the size. The most important effect of the shape, however, is the degree of concentration of the run-off from the several portions of the watershed. Here again, a study of the physical data for each tributary area and the relation of such areas to each other will enable a fairly reliable conclusion to be reached.

The remaining characteristics are more difficult to analyze and few attempts have been made to distinguish between the

effects due to each of them. Formulas have been developed in which slope, perviousness of soil, and surface covering are represented by separate factors; but these formulas have not been used to any great extent because of the additional cost involved in obtaining data and because of the lack of sufficient data to assume satisfactory values for the several factors.

A series of experiments were made to determine the effect of the three last-mentioned characteristics by the engineering staff of the Miami Conservancy District[1] in connection with the Miami flood-control project, and because of their results are worthy of a brief presentation.

Four plots, 5 feet square, were located in an orchard at Moraine Park, which is about 5 miles north of Dayton, Ohio. Two of the plots were on level ground about 10 feet apart, and two were on a hillside where the slope of the surface was about 18 feet in 100 feet. One of the plots in each set was covered with a heavy, blue-grass sod, and the other plot was bare, the sod having been removed. The upper 2 feet of soil was a yellow sandy loam containing some clay and gravel, and was underlaid by a mixed sand and gravel of glacial origin. The plots were separated from the adjoining ground by 12-inch corrugated-iron strips imbedded 8 inches in the soil. Care was taken not to disturb the ground inside the plots and to prevent leakage at the corners. An iron tank to catch the surface run-off was set in the ground just outside the lower corner of each plot, and connected to the inside of the plot by a 3-inch sewer pipe. At first the run-off from actual rainfall was measured, a rain gage near each set recording the rainfall; later artificial rainfall was applied with garden sprinkler cans at a constant rate until the surface became saturated, after which the rate was varied through a range corresponding to actual rainfall conditions.

Some of the more general conclusions reached in regard to the relation of rainfall and run-off in the Miami Valley are as follows:

1. That variations in surface slope are of much less importance as affecting run-off than are variations in vegetable cover.

2. That intensity of precipitation has an important effect on the occurrence and amount of surface run-off.

3. That the rate of surface run-off increases as the rate of precipitation increases, the former being directly proportional to the latter when the surface soil is saturated.

[1] *Technical Report*, part VIII, Miami Conservancy District.

4. That the rate of percolation, when the surface soil is saturated, increases as the rate of rainfall increases, the variation being according to a straight line equation and the rate of increase being proportionally greater for loose, loamy soils than for heavy, clay soils.

5. That cultivation has a relatively important effect in reducing the amount of surface run-off.

6. That the total retention during storms is greater in the summer than in the winter.

7. That the amount of water absorbed by the upper 2 feet during a given storm is greater under sod surfaces than under bare soil surfaces.

8. That appreciable surface run-off frequently occurs during intense summer storms when the upper 6 inches of soil are not nearly saturated.

9. That surface run-off does not occur during some less intense storms even though the ground is saturated.

10. That water can percolate through the 2-foot layer of surface soil on the bare soil plots at Moraine Park when the ground is saturated but not frozen at a rate as great as 0.25 inch per hour.

11. That water can be absorbed by the bare soil at times when the soil is unusually dry at a rate as great as 1 inch per hour for intervals as long as 30 minutes.

12. That water cannot be absorbed by the bare soil at any time, no matter how dry it is, at a rate as great as 3 inches per hour for periods as long as 5 minutes.

CAUSE OF FLOODS

Floods are the result of a favorable combination of precipitation and watershed characteristics. The three more common combinations of conditions are: (1) intense precipitation over small, hilly watersheds; (2) heavy precipitation of several days' duration over large watersheds; and (3) warm spring rains falling on accumulated snows.

Case 1.—As pointed out in the chapter on precipitation, intense rainfall can occur only over comparatively small areas, and when such a precipitation is centered over a small watershed, the inevitable result is a high surface run-off which reaches the main channel faster than it can be discharged, resulting in a flooding of the low lands along the stream. The seriousness of such a flood will depend first of all on the intensity and duration of the precipitation, and second on the surface slopes. The nature of the surface soil and the surface covering are of minor importance, because no soil except sand, even though it be dry, can absorb water at a sufficient rate to make any appreciable difference in the run-off, and the storage capacity of a soil covered

with vegetation is soon exceeded. Intense precipitation is sometimes called a "cloudburst." There seems to be a relation between topography and the intensity of cloudbursts, those occurring in mountainous regions being more frequent and more violent than those over more level country. As examples of floods of this type, the following are cited.

On the morning of June 22, 1902, the entire town of Keystone,[1] West Virginia, was flooded to a depth of 3 feet or more by an intense precipitation over the watershed of Elkhorn Creek which has an area of 44 square miles above Keystone. Between 5 and 6 inches of rain fell in a period of 14 hours. The rise of the Creek at Keystone was 4 feet in 15 minutes, and in 2 hours the stream rose from 2 or 3 feet to 20 feet above low water, causing the loss of about 50 lives and of property valued at $1,250,000. The maximum rate of run-off was estimated at 1,360 second-feet per square mile. The slopes of this watershed are steep and the surface soil is largely rock.

An intense rainfall of the cloudburst type occurred on the night of May 27, 1922, over the Templeton Gap watershed, and caused much damage to property in and near Colorado Springs,[2] Colorado, which is about 5 miles southwest of Templeton Gap. The watershed has an area of 7.1 square miles and is composed of steep, rocky slopes with a small amount of agricultural land. At Colorado Springs the precipitation was only 0.5 inch, but above Templeton Gap 7.0 inches was caught in a pail standing in the open. The duration of the storm was about 3 hours.

A rainfall of 5 inches in 2 hours at Hot Springs,[3] Arkansas, on the afternoon of May 14, 1923, preceded by a rainfall of 2.5 inches earlier in the day, caused a flooding of the city and a damage estimated at $1,000,000. The watershed area is 4 square miles and the small stream which drains it flows through the center of the city through a closed conduit under the principal street. This was the third flood in Hot Springs since the creek was covered over.

Case 2.—Intense precipitation has little effect on a large watershed since it covers a small portion only, and since the channel of the main stream is larger and better able to discharge the run-off. For the large watershed it is the excessive rains lasting

[1] *Eng. News*, Sept. 7, 1902.
[2] *Eng. News-Record*, p. 921, Nov. 30, 1922.
[3] *Eng. News-Record*, p. 922, May 24, 1923.

several days or a series of successive storm periods that cause floods.

The watershed characteristics of principal importance in this case are the imperviousness of the soil and the topography. The more pervious the soil, the greater is its power of retention, and the longer the time required to saturate it. Once the soil is saturated, however, there is little difference in the run-off from a pervious and an impervious area. For this reason, the floods over large watersheds occur generally in the spring when the moisture content of the soil is high.

Since, during a storm period of 4 or 5 days, all portions of a watershed of from 5,000 to 10,000 square miles are contributing simultaneously to the flood flow of the main stream, the shape of the watershed is of secondary importance so far as the high-water stage is concerned, although it may effect the duration of the flood period and the total flood run-off.

An excellent example of a flood due to excessive precipitation is that of Mar. 23 to 27, 1913, in the Miami River Valley in Ohio, which caused the loss of 360 lives and over $100,000,000 in property. During the 5-day period, 11 inches of rain fell over a strip about 10 miles wide extending in a northeasterly direction across the northern third of the watershed, decreasing to 9 inches at the northern end, and to 8 inches at the southern end of the watershed. The average rainfall was 9.6 inches, of which 4.11 inches fell on the third day of the storm period. The watershed is about 120 miles in length and 70 miles in maximum width and contains about 4,000 square miles. The country is rolling, with short, steep slopes leading to numerous little brooks. The soil is a mixture of clay, sand, and gravel and is practically all under cultivation. Some 2,000 miles of ditches have been dug to drain the land.

The maximum flood discharge a short distance above the mouth of the Miami River was 98 second-feet per square mile. The maximum run-off was 571 second-feet per square mile from the Honey Creek watershed which has an area of 52 square miles.

Case 3.—When a large amount of snow accumulates on a watershed during the winter months, there is always danger of serious spring floods. If the rise in temperature in the spring is gradual, the snow will melt slowly and intermittently and the run-off will be discharged without taxing the capacity of the channel. On the other hand, a warm spring rain falling on

accumulated snow is almost sure to cause flooding of the low lands.

The floods in most of the rivers of Maine during the first week of May, 1923, were due to this cause. Snow surveys on the Kennebec and Penobscot headwaters showed about 50 per cent more snow than normal, and the floods which occurred were predicted several weeks in advance. The average depth of snow on the Kennebec watershed above Waterville, an area of 4,270 square miles, was 36 inches; and on the Penobscot watershed above West Enfield, an area of 6,600 square miles, it was 24 inches. On the night of April 28, a general rain occurred over the state, and the Augusta rain gage collected 3.25 inches of water. This was followed on the night of April 29 by a second storm of 1.15 inches. These rains melted nearly all the snow, and the resulting floods caused a property damage estimated between three and four million dollars. The Kennebec, Penobscot, Union, and St. Croix watersheds were particularly affected.

ESTIMATION OF FUTURE RUN-OFF

The importance of estimating correctly the probable magnitude of future floods cannot be emphasized too strongly. It is the fundamental assumption upon which the success of the project depends, and, therefore, should not be decided until all the available meteorological data have been thoroughly examined and the physical characteristics of the watershed investigated. Also the degree of protection warranted must be carefully considered to determine whether the engineering works should be designed to afford protection against the maximum possible flood which may occur once in several hundred years, or against those which may occur with a frequency of once in 10, 25, or 50 years. Because of its importance, the engineer should not allow the impatience of his clients to see construction work started to influence him in deciding this question with undue haste; but he should insist on taking sufficient time in the preliminary investigation to collect all available data, to make any surveys and discharge measurements which will be of assistance, to estimate the damages which will result from a serious flood and the expenditures which are justified for flood protection and drainage work, to weigh carefully all the data thus collected, and finally to arrive at a conclusion based on facts and sound premises.

In most branches of engineering, the engineer can safeguard his work by introducing factors of safety of 3 or 4 to cover possible error in his fundamental assumptions; but in flood control projects, which are based on much more inadequate data, a factor of safety greater than about 1.5 is impracticable because of the additional cost.

Flood-discharge Formulas.—A large number of empirical formulas have been developed for estimating flood run-off. In general such formulas apply to certain streams and areas which are subject to the same rainfall conditions, and in which

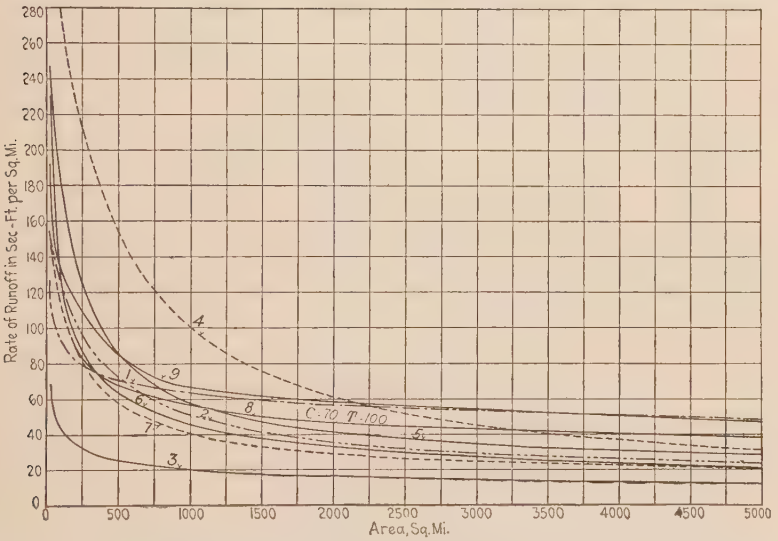


FIG 41.—Graphical comparison of run-off formulas for large watersheds.

the watershed characteristics are similar. In most of them the discharge is some function of the drainage area, which is the only variable, the other factors being taken account of by constants. A few flood-flow formulas for drainage areas over 200 square miles are given in Table IX and represented by curves in Fig. 41. When these formulas are compared they are found to give results which vary widely, and it is quite evident that a general formula applicable to the entire country is impossible.

Mr. Fuller's formula[1] differs from the others in that it contains the time element or frequency of occurrence. In studying the

[1] FULLER, W. E., "Flood Flows," *Trans. Amer. Soc. Civ. Eng.*, vol. 77, p. 564.

TABLE IX.—FLOOD RUN-OFF FOR WATERSHEDS OVER 200 SQUARE MILES IN AREA

Author	Formula	Conditions
1. Fanning	$q = 200 \sqrt[6]{\frac{1}{M}}$	New England streams.
2. Murphy	$q = \frac{46{,}790}{M + 320} + 15$	Northeastern United States (areas up to 10,000 square miles).
3. Cooley	$q = 200 + \sqrt[3]{\frac{1}{M}}$	Mississippi Valley, vicinity of St. Louis. Frequency once in 6 to 10 years.
4. Kuichling	$q = \frac{127{,}000}{M + 370} + 7.4$	Mohawk River. Rare floods. Areas up to 5,000 square miles.
5. Kuichling	$q = \frac{44{,}000}{M + 170} + 20$	Mohawk River, occasional floods.
6. Bremner	$q = \frac{3{,}000}{3 + 2\sqrt{M}}$	C. B. & Q. R. R. for waterway openings.
7. Harman	$q = \frac{30{,}000}{M + 200} + 15$	Southern Illinois streams.
8. Fuller	$q = q_{\text{ave. annual}} (1 + 0.8 \log T)$ $q_{\text{ave. annual}} = \frac{C}{M^{0.2}}$	General.
9. Metcalf and Eddy.	$q = \frac{440}{M^{0.27}}$	

NOMENCLATURE

q = Maximum flood discharge in cubic feet per second per square mile.
M = Drainage area in square miles.
T = Number of years in period considered.
C = Coefficient varying with stream.

run-off from 119 rivers of the United States having run-off records of 10 years or more, Mr. Fuller found that for all the streams there was a fairly similar relation between the magnitude of the large floods and of the average annual floods. For 17 streams, this ratio was between 1.17 and 1.50; for 66 streams, between 1.5 and 2.0; for 31 streams, between 2 and 3; for 5 streams, between 3 and 4; and for one stream it was nearly 5. For each stream the ratio of the maximum flood each year to the average annual flood was computed. The records of all the streams were then combined and treated as one continuous record 1,672 years in length for one stream. The combined ratios were recorded in the order of their magnitude and the frequency of various ratios computed. From these data, the formula, $R = 1 + 0.8 \log T$, was developed, in which T is the frequency in years and

R is the ratio of the greatest flood to be expected in the time T and the average annual maximum flood. The ratios for frequencies from 1 to 1,000 years are given in Table X.

TABLE X.—RATIO OF LARGEST FLOOD TO AVERAGE ANNUAL FLOOD FOR DIFFERENT FREQUENCIES

Frequency T in years	Ratio of largest flood to average annual flood	Frequency T in years	Ratio of largest flood to average annual flood
1	1.00	50	2.36
5	1.56	100	2.60
10	1.80	500	3.16
25	2.12	1,000	3.40

Mr. Fuller also developed the formula, $q = \frac{C}{M^{0.2}}$, for computing the average annual flood. In this formula, M is the size of the watershed in square miles and C is a coefficient varying for each section of the United States to allow for different watershed characteristics and rainfall conditions.

The use of run-off formulas is dangerous unless their limitations are thoroughly understood and allowed for, and even then too much confidence cannot be placed in them. The results obtained from them must be considered approximate only and used chiefly as a check on computation based on more reliable data. All the formulas, however, emphasize one fundamental fact, namely, that the rate of run-off decreases as the watershed area increases.

Rainfall Studies.—Since precipitation is the principal factor which causes floods, it is evident that the first step in estimating flood run-off is a study of rainfall records. The size and shape of the watershed will determine the character of the storms to be studied. For small watersheds—up to about 100 square miles in area—intense precipitations lasting from a few hours to a day are the ones to be examined; while for larger areas, excessive precipitations lasting several days are the ones which are conducive to flood conditions.

Time of Concentration.—Time of concentration or collection is an important factor. Maximum flood conditions exist only when

all portions of a watershed are contributing their maximum flow to the discharge at the point under consideration. For very small, steep watersheds, only a fraction of an hour is required for the run-off from the extreme portions to reach the lower end. As the watershed becomes larger, more and more time is required for this condition to prevail, and hence storms of longer duration must be investigated. The shape of the watershed affects the time of concentration materially and must be given due consideration.

The time of concentration depends upon all the watershed characteristics and upon the amount of precipitation. The best method of determining it is to choose some storm, which occurred when the ground was saturated, and find as nearly as possible the amount of the precipitation and the time required for the waters to concentrate. The greater the volume of flow, the greater will be the velocity and the shorter the time of concentration. If data for past storms are not available, the time of concentration can be approximated by assuming the time required for the water flowing over the surface to reach the tributary channels and estimating the time required for the water from the tributaries to reach the main stream and thence to the place under consideration. The cross-section and gradient of the channels can be measured and the approximate velocity computed by formula. At this stage of the investigation approximate values are sufficient since the only use to be made of them is to determine the approximate duration of the storms to be studied in detail.

Rainfall Records.—As pointed out in the chapter on precipitation, intense rainfall of the cloudburst type is likely to occur anywhere; therefore, in the determination of rainfall over small watersheds, the records of all such storms should be consulted. For storms up to 2 hours' duration, the curves in Figs. 36 to 39 give as reliable data as it is possible to obtain from the records of the past; for storms up to 24 hours' duration, the curve in Fig. 40 can be used. The curves on the isopluvial charts given in Figs. 9 to 32 also give data which can be used for small watersheds for storms of from 1 to 6 days' duration for varying frequencies. In using these charts for a particular area, frequency curves should be drawn by plotting the pluvial indices for the quadrangle containing the area against the frequency of occurrence. For larger watersheds—up to 5,000 to 10,000 square miles—the records of storms of a duration sufficient to produce concentration

of flow should be obtained. Not only must those storms which have taken place over the watershed in question be considered, but also those over near-by watersheds which might just as readily have occurred over the given watershed.

The records of rainfall can be obtained from the Climatological Data published by the United States Weather Bureau. The data for each storm to be studied should be tabulated as shown in Table XI.

Storm Maps.—In order to study each of the storms intelligently and to compare them, a storm map of each is necessary. A watershed map, drawn to a convenient scale, must first be prepared, showing the boundary of the watershed, all streams, and the location of all observing stations within and adjacent to the watershed. The 1:500,000 scale maps issued by the State Geological Surveys are the most reliable and suitable ones to use as a base for areas above 1,000 square miles; while the quadrangle maps drawn to a scale of 1:62,500 are best for smaller areas. The watershed boundary can be drawn on such maps with an accuracy of about 98 per cent. By the use of a pantograph, the desired data can be reproduced to a more convenient scale for further study. Since a number of these maps will be needed, a tracing should be made and from it as many white prints as may be needed.

The rainfall at each station for the first day of the storm is marked on this map and 1-inch isohyetals, or lines of equal rainfall, are drawn by interpolation in the same manner as contours are located on a topographic map. Similar maps are constructed for each day of the storm and for the accumulated rainfall up to the end of each day of the storm period. From these maps the distribution of the precipitation over the watershed throughout the period can be readily studied, and the average rainfall computed. This is done by: (1) finding the area of the watershed between adjacent isohyetals with a planimeter; (2) multiplying each area by the average rainfall within the area; (3) adding the products thus obtained; and (4) dividing the sum of the products by the sum of the areas; the quotient will be the average rainfall over the watershed. The areas may be obtained in any unit, since they represent the relative weights to be assigned the different rainfalls. Where the rainfall for any day is small, it may be necessary to draw the ½-inch and in some cases the ¼-inch isohyetals. If desired, the watershed can

be subdivided into tributary watersheds and the average rainfall over each determined.

By constructing such maps for a number of storms and computing the average rainfalls, the records are converted into forms which can be easily compared, and the approximate frequency of storms of a given intensity determined, provided the record is of sufficient length.

Illustrative Example.—To indicate more clearly the procedure in studying rainfall data, the Kaskaskia River Watershed in

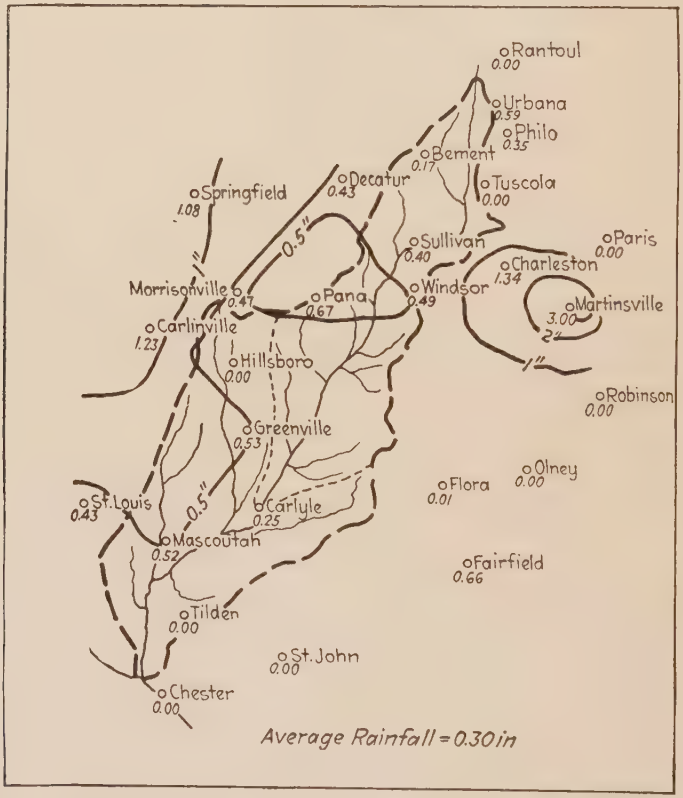


Fig. 42.—Rainfall on Kaskaskia River Watershed, Illinois, on May 3, 1908

southern Illinois, with an area of 5,670 square miles, has been taken as an example. A study of the rainfall records of the United States Weather Bureau shows that storms which caused serious flooding of the bottom lands in this valley since 1870

occurred in 1875, 1882, 1898, 1907, 1908, 1911, 1913, and 1915. Of these storms, that of May 3 to 7, 1908, caused the most disastrous flood and, therefore, it will be chosen for detailed study.

The rainfalls on each day of the storm period at 8 observing stations within, and 21 stations adjacent to, the watershed are given in Table XI. From these data, the storm maps given in Figs. 42 to 47 were prepared. Figure 42 gives the isohyetals for

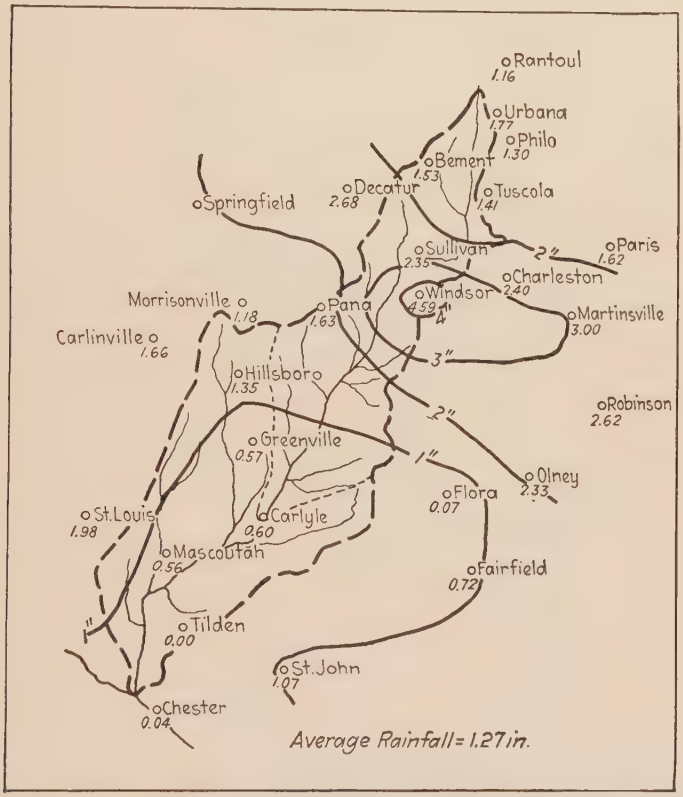


FIG. 43.—Rainfall on Kaskaskia River Watershed, Illinois, on May 3 and 4, 1908.

May 3, and Figs. 43 to 46 show the accumulated rainfall up to the end of each day of the storm period. The average rainfalls for each day from May 3 to 7 were 0.30, 0.97, 1.80, 1.40, and 1.10 inches, respectively, the total average rainfall for the 5-day period being 5.57 inches.

This storm was not centered over the Kaskaskia River Watershed, and had it been, a larger average rainfall would have

resulted. To determine what this value would have been, the isohyetals for the entire storm as shown in Fig. 46 were superimposed on the watershed as shown in Fig. 47 so as to give maximum rainfall conditions, and the average rainfall computed to be 6.21 inches. This would have been the result if the storm had occurred 50 miles to the southwest.

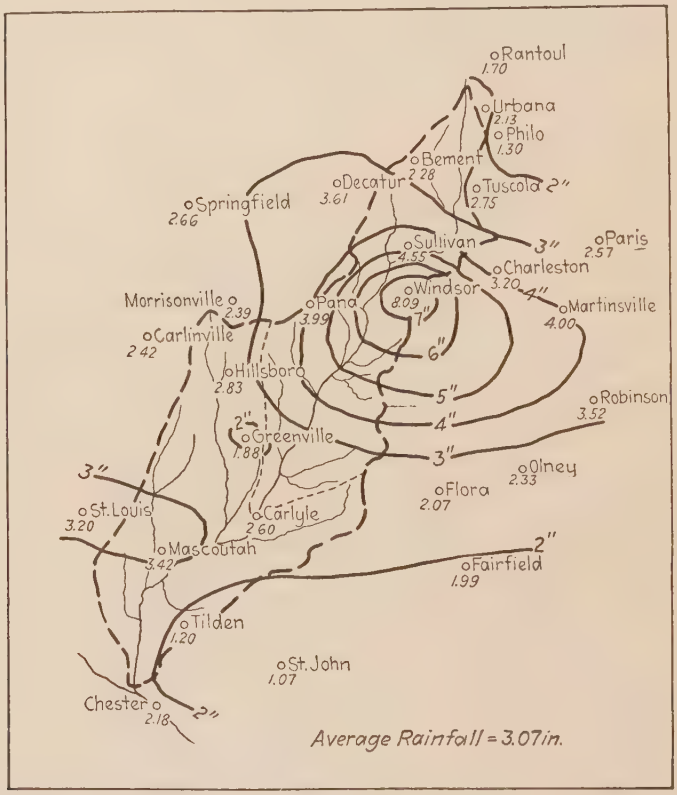


FIG. 44.—Rainfall on Kaskaskia River Watershed, Illinois, on May 3–5, 1908.

While the storm of May 3 to 7, 1908, caused the most serious flooding of any storm of record over the Kaskaskia Watershed, it is not the maximum storm which has occurred in this general region. That of Mar. 23 to 27, 1913, which caused so much damage in Ohio, was much greater. This storm was centered over the Miami and Scioto watersheds, but its borders reached to the Kaskaskia Watershed and was the cause of the 1913 flood

mentioned above. The center of this storm might just as readily have fallen on the Kaskaskia Watershed as upon the Miami, hence it must be considered in determining the maximum rainfall over the former area. This storm is shown superimposed on the Kaskaskia Watershed in Fig. 48, and would have caused an average rainfall of 8.92 inches. Whether or not this is the maximum storm which could occur is a question. Certainly a

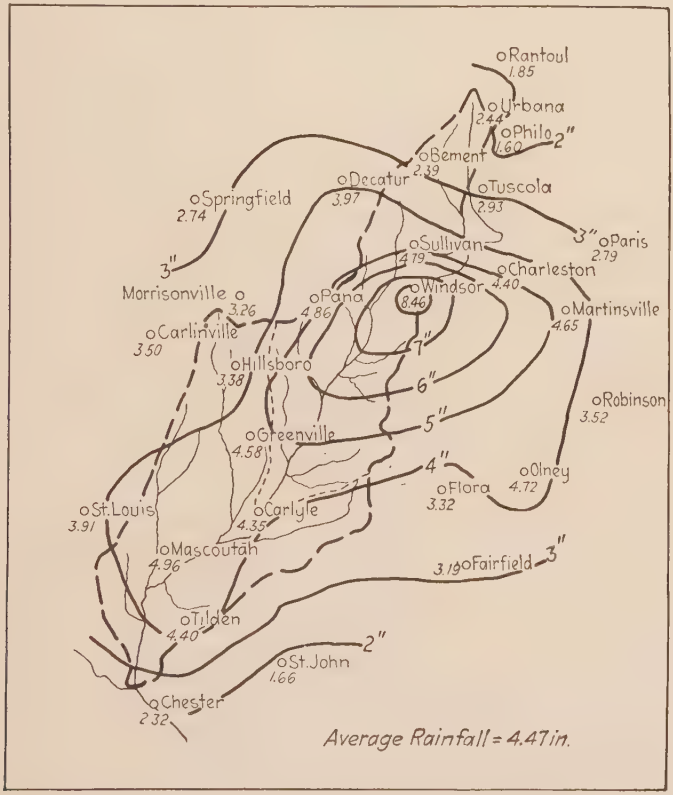


FIG. 45.—Rainfall on Kaskaskia River Watershed, Illinois, on May 3–6, 1908.

storm of this magnitude is not likely to occur oftener than once in several centuries. After an exhaustive study, the engineers of the Miami Conservancy District concluded that a storm from 15 to 20 per cent greater than that of 1913 might occur at some time in the distant future. A storm 20 per cent greater than that of 1913 would cause an average rainfall of 10.70 inches over the Kaskaskia Watershed in a 5-day period.

TABLE XI.—RAINFALL DATA FOR STORM OF MAY 3 TO 7, 1908, OVER KASKASKIA RIVER WATERSHED

Station	May								
	3	4	5	6	7	3–4	3–5	3–6	3–7
Bement	0.17	1.36	0.75	0.11	1.50	1.53	2.28	2.39	3.89
Carlinville	1.23	0.43	0.76	1.08	0.65	1.66	2.42	3.50	4.15
Carlyle	0.25	0.35	2.00	1.75	0.70	0.60	2.60	4.35	5.05
Charleston	1.34	1.06	0.80	1.20	0.14	2.40	3.20	4.40	4.54
Chester	0.00	0.04	2.14	0.14	2.04	0.04	2.18	2.32	4.36
Decatur	0.43	2.25	0.93	0.36	0.72	2.68	3.61	3.97	4.69
Fairfield	0.66	0.06	1.27	1.20	0.47	0.72	1.99	3.19	3.66
Flora	T	0.07	2.00	1.25	1.32	0.07	2.07	3.32	4.64
Greenville	0.53	0.04	1.31	2.70	0.48	0.57	1.88	4.58	5.06
Hillsboro	0.00	1.35	1.48	0.55	1.88	1.35	2.83	3.38	5.26
Hoopeston	0.00	0.81	0.16	0.10	1.61	0.81	0.97	1.07	2.68
Martinton	0.09	1.28	0.17	1.30	0.50	1.37	1.54	2.84	3.34
Martinsville	3.00	0.00	1.00	0.65	4.00	3.00	4.00	4.65	8.65
Mascoutah	0.52	0.04	2.86	1.54	0.50	0.56	3.42	4.96	5.46
Morrisonville	0.47	0.71	1.21	0.87	0.66	1.18	2.39	3.26	3.92
Olney	0.00	2.33	0.00	2.39	T	2.33	2.33	4.72	4.72
Pana	0.67	0.96	2.36	0.87	1.20	1.63	3.99	4.86	6.06
Paris	0.00	1.62	0.95	0.22	3.42	1.62	2.57	2.79	6.21
Philo	0.35	0.95	0.00	0.30	2.05	1.30	1.30	1.60	3.65
Rantoul	0.00	1.16	0.54	0.15	1.68	1.16	1.70	1.85	3.53
Robinson	0.00	2.62	0.90	0.00	4.15	2.62	3.52	3.52	7.67
St. John	0.00	1.07	T	0.59	0.80	1.07	1.07	1.66	2.46
St. Louis	0.43	1.55	1.22	0.71	0.02	1.98	3.20	3.91	3.93
Springfield	1.08	0.88	0.70	0.08	0.17	1.96	2.66	2.74	2.91
Sullivan	0.40	1.95	2.20	0.24	1.56	2.35	4.55	4.79	6.35
Tilden	0.00	0.00	1.20	3.20	0.05	0.00	1.20	4.40	4.45
Tuscola	0.00	1.41	1.34	0.18	1.74	1.41	2.75	2.93	4.67
Urbana	0.59	1.18	0.36	0.31	1.74	1.77	2.13	2.44	4.18
Windsor	0.49	4.10	3.50	0.37	1.50	4.59	8.09	8.46	9.96

A study of Fig. 33 shows that the maximum average rainfall over an area of 6,000 square miles in any of the great rainfalls of record was 10.0 inches for a 5-day storm period, and 11.4 and 11.2 inches for 3- and 4-day storms, respectively. The storm which caused the maximum 3-day precipitation was that of Oct. 4 to 6, 1910, which was centered over the extreme southern portion of Illinois and northern Arkansas; and the storm of Aug. 17 to 20, 1915, which caused the maximum 4-day precipitation, was centered over Arkansas and Texas. The former storm might have occurred over the Kaskaskia Watershed, but it is

extremely doubtful in the case of the latter. These storms occurred in the summer and autumn when the conditions for maximum run-off were unfavorable. It is doubtful if either of them could have happened in the spring when the ground was saturated and the conditions most favorable to large run-off. Hence it is justifiable to assume that the maximum 5-day

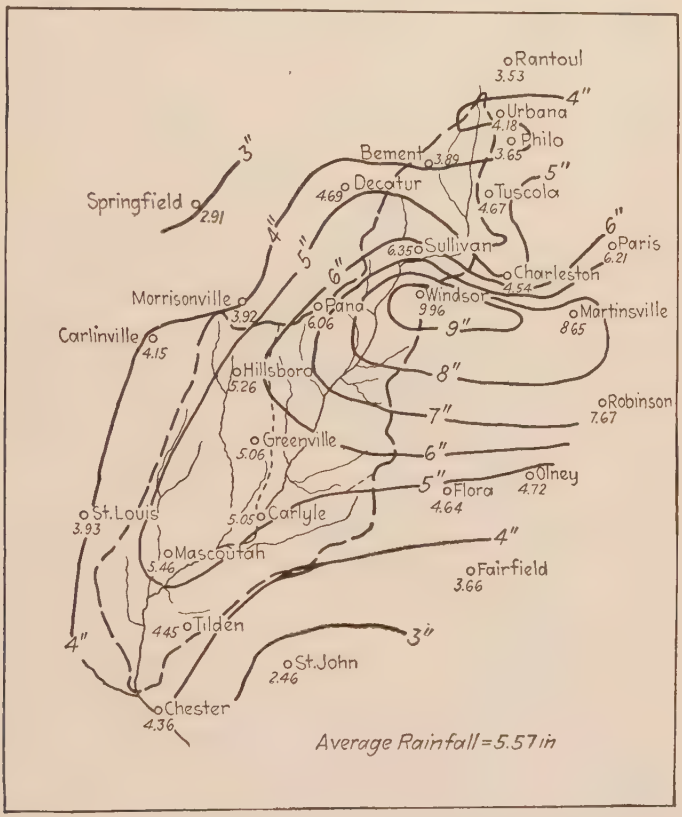


FIG. 46.—Rainfall on Kaskaskia River Watershed, Illinois, on May 3–7, 1908.

precipitation over the Kaskaskia Watershed in a period of several hundred years is not likely to exceed 10 inches.

Run-off Studies.—The amount of flood flow which will result from a given precipitation depends upon the characteristics of the watershed. A rainfall which will cause serious flooding on one watershed may cause no damage on another the same in size but different in other respects. Since it is impossible to

determine, with any degree of accuracy, the relation between rainfall and run-off simply by a study of the watershed itself, some more reliable method must be found. The run-offs which have resulted in the past from rainfalls of different intensities are the best indication of the effect of natural conditions and the safest basis upon which to predict future run-off. All records

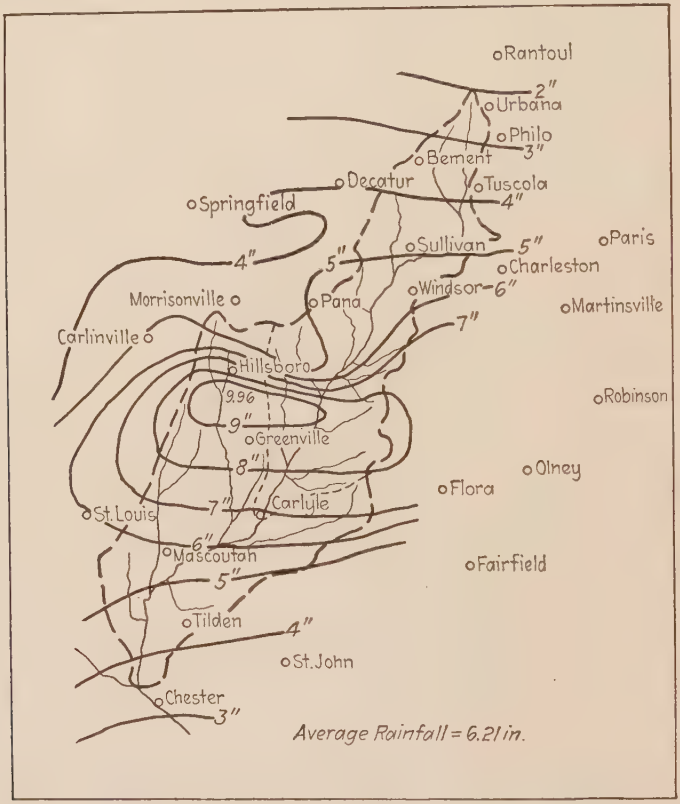


FIG. 47.—Storm of May 3–7, 1908, superimposed on Kaskaskia River Watershed, Illinois, to give maximum rainfall conditions.

of past run-off, therefore, should be obtained and thoroughly studied together with the rainfall records. Unfortunately authentic run-off records covering any considerable periods of time are rare, because of the cost of establishing and maintaining gaging stations for the measurement of stream discharge and of the lack of appreciation of the need and value of such data by the general public. In the eastern and southern parts of the

United States these records are more numerous and extend over longer periods than in the Middle West; and for a few streams, records 100 years long are available. Nearly all of our run-off records have been collected by the United States Geological Survey and are published in the *Water Supply Papers* of that organization.

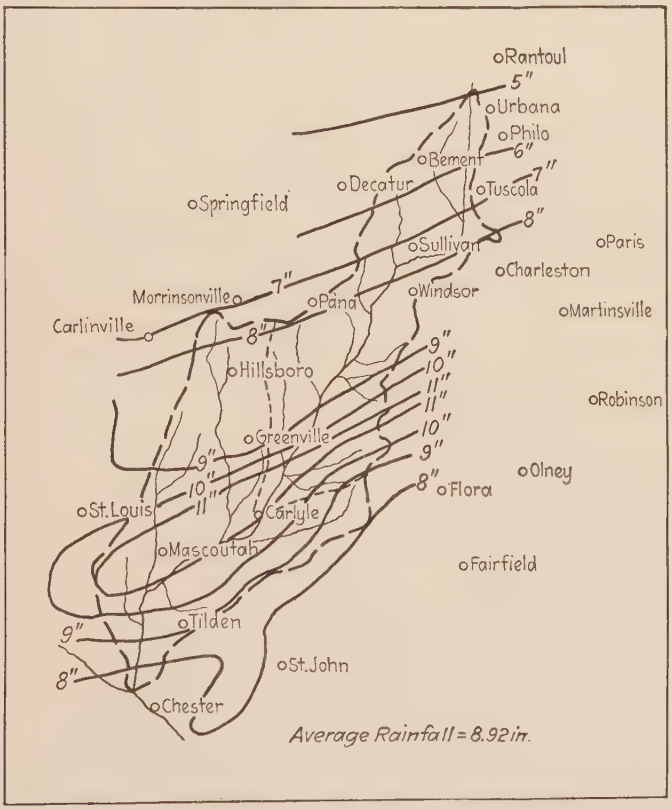


FIG. 48.—Storm of March 23–27, 1913, superimposed on Kaskaskia River Watershed, Illinois.

On those streams where discharge measurements have not been made, frequently staff gages have been established on bridge piers or abutments by municipalities or by private individuals and records kept of the height of the water. From several such records along a stream, the approximate discharge can be computed from a slope formula as described in a subsequent chapter. Or again, high-water marks of the largest floods may

have been established by someone living near the stream, and the marks of such floods on buildings and trees are distinguishable for a considerable period.

The run-off from the larger storms must then be compared with the rainfalls which caused them, and a relation between rainfall and run-off determined for each storm. These may vary considerably even for rainfalls of the same intensity and duration. During the summer when the ground is dry and when the rates of evaporation and transpiration are high, a much lower run-off will result than for a similar storm during the spring when conditions are different. Since it is the worst combination of conditions which cause the greatest floods, only those occurring under adverse conditions should be considered, or else the relation between rainfall and run-off in those cases should be modified

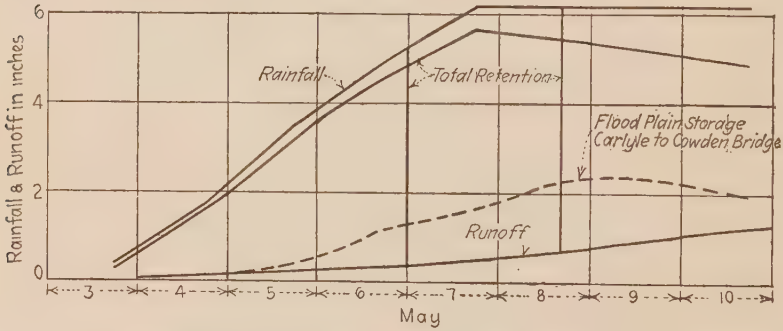


FIG. 49.—Rainfall, run-off and storage curves for the Kaskaskia River Watershed at Carlyle, Ill., for storm of May 3–7, 1908.

to allow for the different conditions. The run-off in second-feet should then be plotted as ordinates against the rainfalls as abscissas and a curve drawn through these points as nearly as may be. Such a curve will approach a straight line at its upper end and the slope of this line is a measure of the combined effect of the watershed characteristics. This curve can be extended somewhat to give the run-off which may be expected from rainfalls of greater magnitude, such as the previously made studies of rainfall have shown to be possible.

Curves should also be drawn for each storm showing the rainfall, run-off, and storage on the watershed for each day of the storm period. These data should be prepared for each place whose run-off records can be obtained. Such curves for the Kaskaskia River at Carlyle are shown in Fig. 49. The drainage

area above Carlyle is 2,680 square miles, and the average rainfalls for each day from May 3 to 7 were 0.39, 1.30, 1.83, 1.44, and 1.21 inches, respectively, the total rainfall being 6.17 inches. This watershed is quite level and the capacity of the river channel is only about 0.20 inch over the watershed in 24 hours. The result is that a large volume of water is held in storage for weeks after the end of the storm. After the storm of May 3 to 7, 1908, the river was out of its banks at Carlyle until June 3. The total retention curve in Fig. 49 is the difference between the rainfall and run-off curves and represents the depth of rainfall held in storage along the flood plain and that absorbed by the ground or retained in shallow depressions over the entire area. The fourth curve represents the depth of rainfall stored in the flood plain between Carlyle and Cowden Bridge. At the crest of the flood on May 8, 2.37[1] inches of rainfall were stored in the valley between Carlyle and Cowden Bridge and a total of 0.70 inch had run off, making a total inflow of 3.07 inches, which was 50 per cent of the total rainfall.

By drawing similar curves for other points on the stream and studying them, the relation between rainfall and run-off at different points along the river can be determined for this storm. By comparing these curves with similar ones for other storms, a fairly satisfactory conclusion can be reached as to the discharge which must be provided for in any plan of flood protection.

The above procedure is based on the assumption that run-off records are available for a sufficient length of time to enable a satisfactory relation between rainfall and run-off to be established. In many instances, such records are lacking and the problem must be solved from another angle. Under these circumstances, watersheds of as similar a nature and as near the given watershed as possible, and for which run-off data are obtainable, must be studied, and a rainfall-run-off relation determined. Since no two watersheds are alike, modifications will have to be made based upon the judgment of the engineer after a careful correlation of the watersheds has been made. It is in these studies that an understanding of the effects of the various watershed characteristics is particularly valuable. While the latter method of determining run-off is often far from satisfactory and contains many chances of error, it is the only course to follow. At the

[1] "Reclamation of Lands Subject to Overflow in the Kaskaskia River Valley," Report of Rivers and Lakes Commission of Illinois.

beginning of an investigation of this kind, gaging stations should be established and as many discharge measurements obtained as the limited time will permit.

Valley Storage.—The amount of temporary storage afforded by the bottom lands along a stream has a decided effect upon the rate of run-off and the height of floods. This is well illustrated by the May, 1908, flood on the Kaskaskia River. The flood plain at Cowden Bridge is about ½-mile in width; at Vandalia, which is 24 miles south, its width is about 3.5 miles; and at Carlyle, 30 miles farther south, it is from 2 to 4 miles. At the peak of the flood on May 8, the water in storage amounted to 339,000 acre-feet or over ten times the daily discharge. The maximum rate of inflow into the valley was about 1.10 inches in 24 hours, or about five times the capacity of the channel.

Small Watersheds.—The maximum run-off from small watersheds is much more difficult to predict satisfactorily, since natural conditions play a more important part than in the case of large areas. The engineer is handicapped also by the lack of authentic data. Usually the only information to be had is the uncertain elevation of high water and possibly the rainfall measurement at some station outside the watershed where the intensity of precipitation for a given storm is undoubtedly different from that over the area under investigation, especially during those erratic storms which generally are the cause of floods on small areas. A careful study and correlation of the storms over other more or less similar watersheds must be undertaken.

The maximum rate of run-off from a number of small areas in the United States, as collected by Ivan E. Houk,[1] are listed in Table XII and plotted in Fig. 50. The three curves drawn from these data give a basis for maximum, general, and minimum design of bridge openings. Mr. Houk discusses the use of these curves as follows:

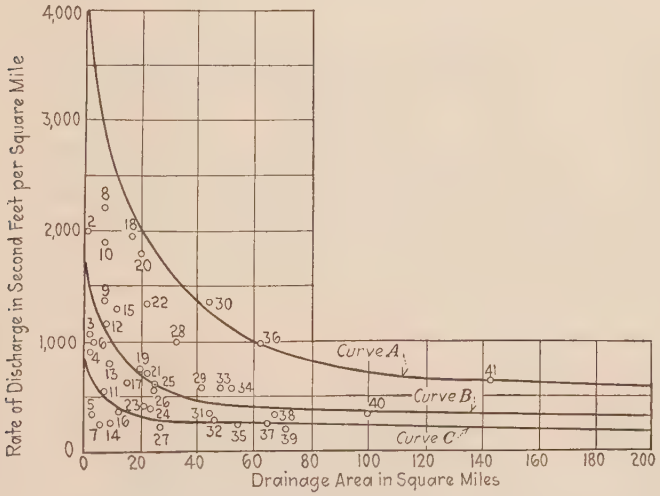
> It is manifestly impracticable to design bridge openings for the run-off rates indicated by Curve A, except in rare instances where the channel cross-section is unusually large, or where the damage which would result from the failure of the structure would be so great as to warrant the unusual cost. The proportion of the maximum possible rate which should be used in design varies with the relative cost of the structure, the extent of the damage in case of failure, and the size of the drainage area. Other things being equal, the larger the drainage area

[1] *Eng. News-Record*, p. 1073, June 29, 1922.

the larger the proportion of the maximum possible flood rate to be considered. The reason for this is that while the maximum flood of a thousand years is not more than a third or a half larger than the maximum of a hundred years in the case of the larger drainage areas (see Miami Conservancy District's *Technical Report*, Part V, page 273), it may in the case of extremely small areas be several times as great.

Curve B indicates the rates of discharge which it would be well to design for whenever it is financially possible to do so. Curve C indicates the minimum rates to be used.

While these curves are valuable, they should not be used without first studying the characteristics of those watersheds comparable to the one under investigation, and making any modifications which may be justified.



(Compiled by Ivan E. Houk, *Engineering News-Record*, June 29, 1922.)

FIG. 50.—Rates of discharge from small drainage areas.

Flood Run-off from Drainage Districts.—The drainage engineer in designing ditches to carry off the water from those flat agricultural areas which are not provided with adequate drainage by nature must estimate the rate at which the water must be removed to prevent damage to crops during excessive storms.

The size of drainage districts varies from a few hundred acres to several hundred square miles. The average size of the 1,043 districts in the State of Illinois, as of September, 1920, was 4,420 acres, and the maximum size was 110,000 acres. Possibly the largest district in the United States is the Little River Drainage District in southeastern Missouri, which contains 488,000 acres.

TABLE XII.—MAXIMUM RATES OF DISCHARGE FROM SMALL AREAS

Reference number	Stream	Location	Drainage area, square miles	Date	Maximum rate in second-foot per square mile	Authority
1	Doker's Hollow	Pennsylvania	0.6	June 1917	4,000	Unpublished data.
2	Mad Creek	Le Roy, N. Y.	1.5	June 1916	2,000	*Eng. Record*, June 24, 1916.
3	Arroyo	Near Pueblo, Colo.	1.8	June 1921	1,060	*Proc. A. S. C. E.*, p. 772, December 1921.
4	Cherryvale Creek	Cherryvale, Kan.	2.0	.....	930	*Trans. A . S. C. E.*, p. 200, 1905.
5	Moraine Run	Moraine City, Ohio	2.6	September 1916	346	*Eng. News-Record*, May 1, 1919.
6	W. Fork Honey Creek	New Carlisle, Ohio	3.5	July 1918	1,000	*Eng. News-Record*, Sept. 22, 1921.
7	Mill Brook	Sherburne, N. Y.	5.0	September 1905	262	*Trans. A. S. C. E.*, p. 564, 1914
8	E. Fork Honey Creek	New Carlisle, Ohio	6.7	July 1918	2,210	*Eng. News-Record*, Sept. 22, 1921
9	Blue Ribbon Creek	Near Pueblo, Colo.	6.7	June 1921	1,360	*Proc. A. S. C. E.*, p. 772, December 1921
10	Cameron Arroyo	Near Pueblo, Colo.	7.3	June 1921	1,900	*Proc. A. S. C. E.*, p. 772, December 1921
11	Sawpit Canyon	Los Angeles, Cal.	7.4	1889	550	Los Angeles Flood Control *Report.*
12	Osteen Arroyo	Near Pueblo, Colo.	7.8	June 1921	1,160	*Proc. A. S. C. E.*, p. 772, December 1921
13	Mill Creek	Erie, Pa.	9.3	August 1915	800	*Eng. Record*, Aug. 14, 1915
14	Mill Brook	Sherburne, N. Y.	9.4	September 1905	241	*Trans. A. S. C. E.*, p. 564, 1914
15	E. Fork Honey Creek	New Carlisle, Ohio	11.8	July 1918	1,290	*Eng. News-Record*, Sept. 22, 1921
16	Eaton Wash	Los Angeles, Cal.	12.5	1914	367	Los Angeles Flood Control *Report*
17	Arroyo	Near Pueblo, Colo.	15.0	June 1921	619	*Proc. A. S. C. E.*, p. 772, December 1921
18	Alazan Creek	Near San Antonio, Tex.	16.9	September 1921	1,950	*Proc. A. S. C. E.*, p. 454, November 1921
19	Ridley Creek	Sycamore Mills, Pa.	20.0	August 1843	750	Unpublished data
20	Willow Creek	Heppner, Ore.	20.0	June 1903	1,800	*Water Supply Paper* 96, U. S. Geol. Survey

21	Apache Creek	Near San Antonio, Tex.	22.0	September 1921	704	*Proc. A. S. C. E.*, p. 454, November 1921
22	Cane Creek	Bakersville, N. C.	22.0	May 1901	1,341	*Eng. News*, Aug. 7, 1902
23	Crum Creek	Delaware County, Pennsylvania	22.0	August 1843	410	Unpublished data
24	River des Peres	St. Louis, Mo.	23.4	August 1915	410	*Eng. News*, Oct. 14, 1915
25	Boggs Creek	Near Pueblo, Colo.	25.0	June 1921	600	Unpublished data
26	Pinal Creek	Globe, Ariz.	25.0	August 1904	560	*Trans. A. S. C. E.*, p. 564, 1914
27	Spring Creek	Piqua, Ohio	27.0	March 1913	210	Miami Conservancy District, *Technical Report*, Part IV
28	Olmos Creek	Near San Antonio, Tex.	32.4	September 1921	1,000	*Proc. A. S. C. E.*, p. 453, November 1921
29	Olmos Creek	Near San Antonio, Tex.	41.0	September 1921	580	*Proc. A. S. C. E.*, p. 453, November 1921
30	Elkhorn Creek	Keystone, W. Va.	44.0	June 1901	1,360	*Eng. News*, Aug. 7, 1902
31	Olmos Creek	Near San Antonio, Tex.	45.0	September 1921	333	*Proc. A. S. C. E.*, p. 453, November 1921
32	Peck Creek	Near Pueblo, Colo.	46.0	June 1921	283	Unpublished data
33	Darby Creek	Delaware County, Pennsylvania	48.0	August 1843	580	Unpublished data
34	Lost Creek	Troy, Ohio	52.0	March 1913	571	Miami Conservancy District, *Technical Report*, Part IV
35	Tawawa Creek	Sidney, Ohio	54.0	March 1913	239	Miami Conservancy District, *Technical Report*, Part IV
36	Chester Creek	Bridgewater, Pa.	62.0	August 1843	1,000	Unpublished data
37	Ludlow Creek	Ludlow Falls, Ohio	65.0	March 1913	266	Miami Conservancy District, *Technical Report*, Part IV
38	Rock Creek	Near Pueblo, Colo.	67.0	June 1921	348	Unpublished data
39	Dry Creek	Near Pueblo, Colo.	71.0	June 1921	225	Unpublished data
40	Sweetwater River	California	100.0	January 1916	330	*Eng. Record*, Feb. 19, 1916
41	Devil's Creek	Viele, Iowa.	143.0	June 1905	650	Recorded estimate

The precipitation to which these areas are subject is the same as that over any other area of equal size in the same general region, and the rainfall studies for small areas previously described apply to them, except that those storms which are likely to occur in the late spring and summer are given principal consideration, since they are the one which affect the time of planting and the growth of crops. The other factors, however, involved in the rate of run-off are different. In the first place, these areas are level or nearly so or else they would not require artificial drainage. In the second place, all the area is under cultivation, or is to be placed under cultivation eventually, which indicates that the soil is free from rock and of a texture which will absorb water readily. Furthermore, drainage channels are provided about every one-half mile for collecting the run-off, and the land is underdrained either by tile or by small ditches every few hundred feet to remove the gravitational water in the soil. Since the top 6 or 7 inches of soil is broken up by frequent plowing, it is highly pervious and will absorb a considerable amount of water before any surface run-off can take place. Plowing also provides furrows which retain the water and reduce surface flow. The subsoil, due to underdrainage, will absorb water more quickly than similar soil which is not underdrained. The tile drains also remove the free water in the soil between rains and thus provide a storage reservoir in the soil which increases the percolation opportunity and reduces the amount of surface run-off. Because of the rapid absorption and large storage capacity of these soils, a heavy precipitation of short duration in many instances results in a light surface run-off. During storms of several days' duration, however, the soil becomes saturated, the surface storage is filled, and large surface run-off takes place, considering the flat slopes of the ground. In this case, the tile drains are contributing their maximum to the flow in the ditches and the rate of run-off is greater than before the drainage system was constructed. This must be true or else the chief purpose of the system, which is to remove the excess water before crops are damaged, would not be attained. Nevertheless, the run-off from these flat areas is necessarily less than that from the normal drainage area of streams. This statement does not apply to districts lying in the flood plains of large streams which are protected by levees from the flood waters of

the steam, but subject to inflow of water from the higher lands outside the district.

Precipitation is undoubtedly the principal factor in the run-off from drainage districts, and consequently the run-off from drainage districts in the south is much greater than from those in the north, regardless of other conditions. The perviousness of the soil is the second factor of importance. The soils of all such areas are comparatively open or else the land would not be suitable for agriculture and the benefits derived from drainage would be less than the cost. These areas may be divided into three general classes: (1) the upland areas of the north central states, where the soils are loams containing varying amounts of silt, clay, sand, and gravel; (2) the low flat lands in the flood plains of large rivers where the soil is composed of alluvium deposited by the streams, mixed with decayed vegetation near the surface; and (3) swamp areas, such as are found in Minnesota and in the Florida Everglades, where the soil is composed of peat and muck. Each of these types of soil will give a different run-off for the same precipitation and the designs for each must be different.

Run-off Curves for Drainage Districts.—A number of run-off curves used or proposed for different sections of the country are given in Fig. 51. Curve 1 was developed by C. G. Elliott[1] for the swamp and other wet lands of the upper Mississippi Valley. It is almost identical with curve 2 which was recommended by John T. Stewart[2] for use in designing drainage ditches in the valley of the Red River of the North in North Dakota. The water capacity of the top 3 feet of typical Red River Valley soil is about 65.5 per cent of its volume, as compared with about 52 per cent for bottom lands in Illinois and Iowa, and a comparatively small rate of run-off is to be expected.

Curve 3 was recommended by Elliott for designing open ditch outlets in Central Illinois as the result of a number of run-off measurements made in 1908 on ditches in Livingston, Ford, and Iroquois counties, where most of the topsoil is a black prairie loam varying in depth from 9 to 18 inches, and the subsoil a firm yellow clay, grading off into porous sand and gravel.

[1] ELLIOTT, C. G., "Engineering for Land Drainage," John Wiley & Sons, Inc., New York, 1919.

[2] STEWART, JOHN T., "Report on the Drainage of Parts of Cass, Trail, Grand Forks, Walsh, and Pembina counties, North Dakota," *Bull.* 189, Office of Experiment Stations, U. S. Department of Agriculture.

Curve 4 was used by A. E. Morgan[1] in estimating the run-off from the flat lands of the main drainage area of the St. Francis River Valley in Northeastern Arkansas. In developing this area later, however, the Morgan Engineering Company found that curve 4 did not give sufficient drainage and curve 5 was used.[2]

Curve 6 was used in estimating the run-off from the flat areas in Washington County, Mississippi, by the Drainage Investiga-

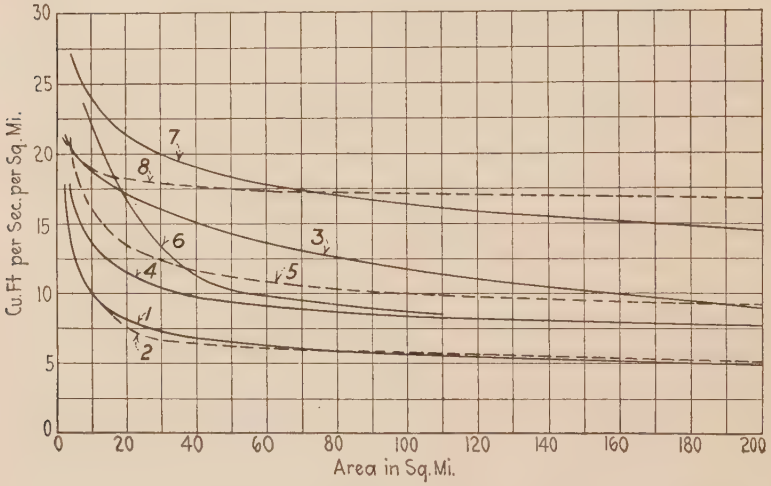


FIG. 51.—Run-off curves which have been used in the design of drainage channels in different sections of the United States.

tions Division of the United States Department of Agriculture,[3] in 1912.

Curve 7 was proposed in 1915 by S. H. McCrory[4] and associates for determining the run-off in the Cypress Creek Drainage District in Desha and Chicot counties, Arkansas. The formula from which the curve is plotted, $q = \frac{35}{\sqrt[6]{M}}$, is a modification of Fanning's formula, $q = \frac{200}{\sqrt[6]{M}}$. The coefficient was determined

[1] MORGAN, A. E., "Report on the St. Francis Drainage Project in Northeastern Arkansas," *Bull.* 230, Office of Experiment Stations, United States Department of Agriculture, 1911.

[2] MEAD, D. W., "Hydrology," p. 588.

[3] *Bull.* 244, Office of Experiment Stations, United States Department of Agriculture.

[4] *Bull.* 198, Office of Experiment Stations, United States Department of Agriculture.

after a series of discharge measurements on the streams within the area and a study of some of the larger drainage districts in the vicinity.

Curve 8 is recommended by F. C. Elliott, chief engineer of the Everglades Drainage District, for use in land drainage work where muck and peat soils are encountered. Mr. Elliott[1] states that an examination of the rainfall records of stations nearest the Everglades since 1895 shows that, if the land had been drained by ditches designed in accordance with the curve, there would have been two general floods in 25 years during which water would have stood on the ground 2 days; and six floods of small area, four of which would have required less than 2 days for their reduction and two would have required 4 days for reducing the water to subsurface conditions. Also that only one locality would have had two local floods in the 25-year period.

While the curves given are valuable, especially for areas in the vicinity of the districts for which they were developed, yet they should not be used blindly as a basis of design, but rather as a check on estimates of run-off made after all data on local rainfall, run-off, and soil conditions have been carefully studied.

Drainage Modulus.—The run-off from drainage districts is frequently expressed in inches of depth of water removed from the area in 24 hours, and this expression is known as the drainage modulus, or sometimes as the drainage coefficient. Sometimes run-off is given in cubic feet per second per acre instead of cubic feet per second per square mile. To simplify the conversion of these units, Table XIII has been prepared giving the relation between the several units.

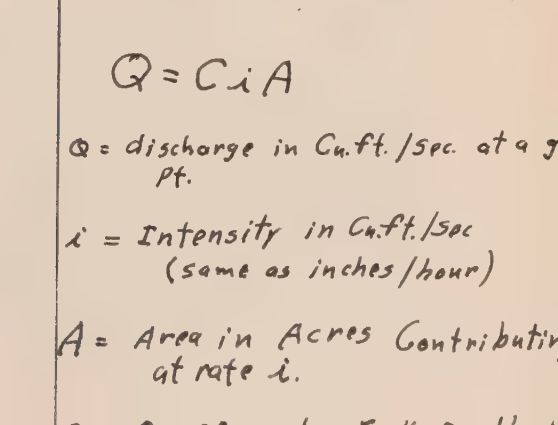
[1] *Eng. News-Record*, p. 157, July 28, 1921.

TABLE XIII.—RELATION BETWEEN RUN-OFF UNITS OF MEASUREMETN

Drainage modulus, inches	Cubic feet per second per acre	Cubic feet per second per square mile
1/16	0.0026	1.68
1/8	0.0052	3.36
3/16	0.0079	5.04
1/4	0.0105	6.72
5/16	0.0131	8.40
3/8	0.0157	10.08
7/16	0.0184	11.76
1/2	0.0210	13.44
9/16	0.0236	15.12
5/8	0.0262	16.80
11/16	0.0289	18.48
3/4	0.0315	20.16
13/16	0.0341	21.84
7/8	0.0367	23.52
15/16	0.0394	25.20
1	0.0420	26.88

Degree of Protection Warranted.—As the result of the rainfall and run-off studies of a given watershed, conclusions are formed as to the probable magnitude of the flood run-off which will occur with frequencies of 5, 10, 25, 50, and 100 years. The question then arises as to what degree of protection is warranted. This question cannot be answered until the value of the property protected and the losses caused by flooding have been investigated. Where the valley is thickly inhabited and the failure of protective works will result in loss of life and the destruction of valuable city property, protection against the maximum possible flood is justified without question. On the other hand, if the only damage from failure is the flooding of agricultural lands and the loss of crops, a much smaller degree of protection is warranted, since it is less expensive to the landowner who pays for the improvement to lose a crop once in 15 or 20 years than it is to build protective works which are likely to prove inadequate once in 25 or 50 years. These are the two extremes and between them lie varying combinations of conditions which call for varying

measures of relief. Aside from the loss of life phase, the question is purely one of economics and can be decided intelligently only after probable losses have been estimated, and engineering plans, affording different degrees of protection, have been prepared and the costs of construction estimated. The subject of flood losses and the various items to be considered are discussed in Chap. I.


Rational Formula
Q = C i A
Q = discharge in Cu. ft. / sec. at a
ft.
i = Intensity in Cu. ft. / sec
(same as inches / hour)
A = Area in Acres Contributi
at rate i.

CHAPTER IV

STREAM DISCHARGE MEASUREMENTS

Obviously the most satisfactory determination of the run-off of a watershed is obtained by actually measuring the discharge of the stream which drains it; and hence it is necessary for the engineer to know the best method of making discharge measurements and how to interpret and use the data after they have been obtained.

A record of the daily discharge of streams for a period of sufficient length to include years of both high and low flow is of primary importance in the design and successful operation of most hydraulic engineering projects. The longer the record the more valuable it becomes, since it will include wider ranges of flow; and usually it is the maximum and minimum flows which govern rather than the average flow. Therefore, it is desirable to measure the discharge of streams before the need for such data is apparent. Appreciating this fact and realizing that this work could not be undertaken to any considerable extent by individuals, the United States Geological Survey, in 1888, started the systematic measurement of the discharge of the streams of the United States. Due to lack of funds, however, its work has been limited to the larger and more important streams, and there remain a large number for which no data are available; and the engineer in private practice is frequently called on to make discharge measurements before he can proceed intelligently with his design. The discharge during flood stages is of special importance in flood protection design.

The methods of making discharge measurements which have been developed by the United States Geological Survey are considered standard and are described in detail in the *Water Supply Papers* issued by that Bureau, together with the discharge data collected.

Discharge.—The discharge of a stream is the volume of water which flows past a given point in a unit of time, and is usually expressed in cubic feet per second, or second-feet. The discharge is the product of the cross-sectional area of the channel and the

mean velocity of flow, and a discharge measurement consists in the determination of these two factors.

The object of discharge measurements is to obtain data from which a rating or discharge curve can be drawn, showing the relation between the discharge and the stage of the stream. Measurements are made at various stages and the results plotted with gage heights as ordinates, and discharges as abscissas. From the curve drawn through the points thus plotted, discharges for stages other than those at which measurements were made can be obtained, and for convenience a table prepared showing the discharge for each tenth or hundredth of a foot change in stage.

Gaging Station.—A gaging station is the place where gage heights of a stream are observed, and at, or near, which discharge measurements are made. Considerable care must be taken in selecting the site for a gaging station in order that the measurements may give reliable results and that the relation between discharge and stage will remain permanent. To secure measurements at extreme stages, a period of several years is generally required; and since the data obtained are treated as though they were secured at the same time, it is essential that the cross-section of the channel at the station and the velocity of flow at given stages remain unchanged. Hence, the requisite conditions of a good gaging station are:

1. The bed and banks of the stream should be stable so that the cross-sectional area at, and below, the station will remain constant, and should be fairly smooth and straight so that there will be no marked cross-currents or eddies to affect the velocity measurements.

2. The slope of the water surface should be the same at all times for the same stage, since the discharge is a function not only of the stage but also of the slope. This condition cannot be fully attained because the slope of a stream when rising is greater than when falling even for the same stage. This change in slope, however, can be minimized by establishing the gaging station just above a point of control such as the crest of rapids, or a permanent ledge across the stream, where the fall at all stages is greater than at the station. A long stretch of river with uniform bottom slope accomplishes the same purpose.

3. The slope and size of the control section should be such as to give as large a change in stage as possible for a given change in

discharge. This relation is known as the "sensitiveness of the station."

4. The location should be such that the cost of installation and operation will be reasonable.

Gages.—The gage is the instrument installed at the gaging station for observing or recording the stage of the stream. They are of two kinds, recording and non-recording.

A common form of non-recording gage, called a staff gage, consists of a graduated rod or plank, firmly attached to a support in the water, with its zero below low-water, so that there will be no negative readings. The staff is read by noting the reading on the gage at the surface of the water. The gage may be vertical and attached to some object in the stream, such as a bridge pier or pile, or it may be inclined and fastened to supports on the bank, in which case the graduations will be governed by the angle of inclination. A very satisfactory and easily constructed staff gage may be made of 2- by 4-inch studding, painted white to keep out the moisture and graduated into feet and tenths by using small brass-headed tacks, one tack for the tenths, two tacks for the half-foot graduations, and three tacks for the feet. Metal numbers, similar to those used in numbering houses, may be used to number the foot graduations. Where there is a considerable range in the stages, two staff gages may be necessary; one fastened by bolts to a bridge abutment, tree, or other permanent support near the bank for medium and high stages, and another farther from the bank—possibly driven securely into the bed of the stream—for low-stage readings. In this case the two gages will need to be referred to the same datum, and should be checked occasionally to guard against errors due to displacement of the lower gage.

Other types of non-recording gages have graduated boards above the water surface, usually attached horizontally to the floor or railing of a bridge, and the stage is read from a marker attached to a movable chain passing over pulleys at the ends of the board. In the float type, a float which rises and falls with the water is attached to one end of the chain and a counterweight to the other to keep the chain taut. In the weight type, a weight attached to one end of the chain is lowered until it comes in contact with the water surface, and the stage read by the marker at the other end. Hook gages of various designs are occasionally used where greater accuracy is desired.

Non-recording gages of the types described are used where it is possible to secure an observer to make the daily readings; but where the station is located in an out-of-the-way place or where the daily fluctuation of stage is irregular, it becomes necessary to install recording gages which require attention only once a week, or in some makes once a month. There are two types of recording gages used, the float gage and the pressure gage. The essential parts of a float gage are: (1) a float—in a stilling box on the bank—which rises and falls with the water surface; (2) a

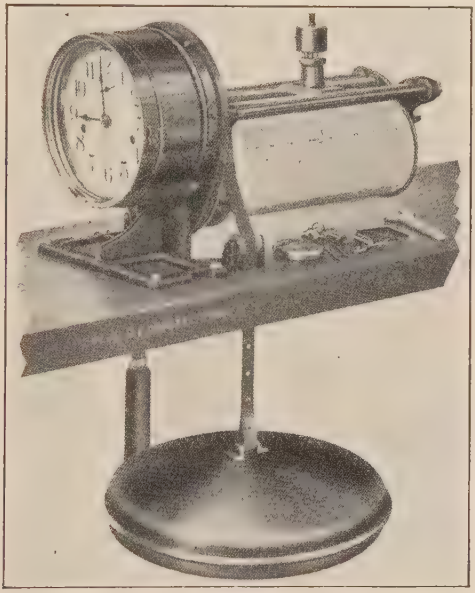


FIG. 52.—Gurley graphic water-stage register.

drum with a record sheet on its surface which is rotated by the action of the float through a chain; and (3) a clock which moves a pencil or pen axially along the drum. The pencil is in contact with the record sheet and produces a curve as the drum rotates and the pencil itself is moved along the drum. This type of gage is illustrated in Fig. 52.

The pressure type of recording gage consists of a "compensator" or of an airtight bulb connected by copper tubing to a pressure chamber which actuates a pen-arm in contact with a circular chart which is revolved by clockwork. The compensator is securely fastened in the bed of the stream below low water,

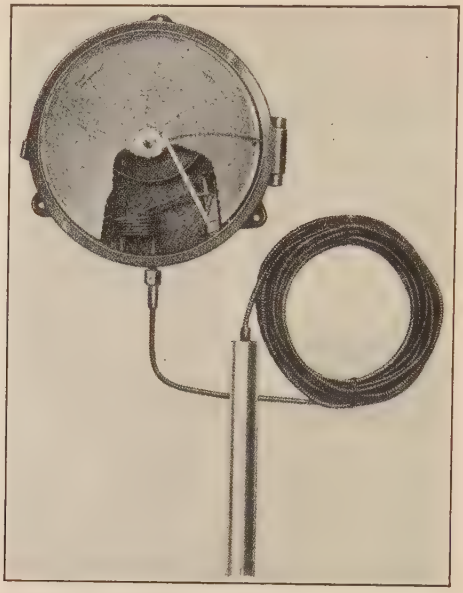


FIG. 53.—Sanborn water-level recorder.

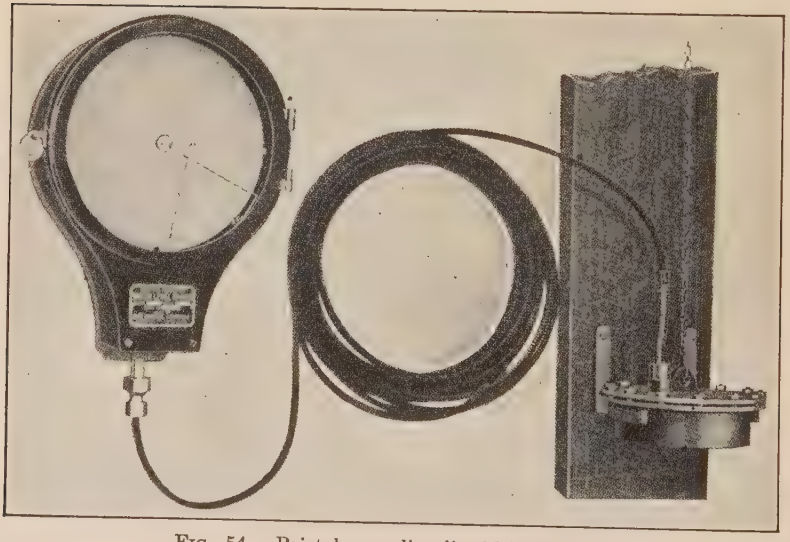


FIG. 54.—Bristol recording liquid-level gage.

and its bottom is the zero of the gage. The weight of the water above the compensator compresses the air in the copper tubing and the pressure chamber, and the movement of a diaphragm on the latter actuates a pen-arm in contact with the record sheet. The Sanborn recording gage is shown in Fig. 53, and the Bristol gage in Fig. 54. Recently there has been placed on the market a gage which registers electrically the stage of a stream at any distance from the gaging station.

Area of Cross-section.—The cross-sectional area of the channel at the place where velocity measurements are made is obtained by taking soundings below the water level at intervals of from 2 to 20 feet across the stream, depending upon its width and regularity of banks and bed, and by using a level for determining the elevations of points on the bank above the water level. A permanent point should be established on the bank above high water to be used as the zero point for all measurements across the stream. In this way, velocity measurements made at different times can be taken at the same points across the stream, and the depths of these points can be quickly computed by simply noting the stage of the water on the gage, provided the bed of the stream remains unchanged. Where the gaging station is at a bridge, permanent marks can be painted on the bridge at intervals of 5 or 10 feet.

From the measurements taken, the cross-section of the channel can be drawn to scale and the area at any stage can be computed either geometrically, *i.e.*, considering the cross-section as made up of a number of trapezoids, or by the use of a planimeter. For a permanent channel, the areas thus obtained can be plotted as abscissas against gage heights as ordinates and an area curve drawn which, used with the mean velocity curve to be described later, provides a check on the discharge curve for stages higher than those at which measurements have been made.

Soundings are made either with a graduated wooden or iron rod with an iron shoe at its bottom to prevent settling in the mud and to give weight to the rod, or with a weight and line. The rod can be used up to depths of about 8 feet, and the soundings made either by wading, for shallow streams, or from a boat held in line by a cable stretched across the stream. The weight and line can be used either from a boat, a car suspended from a cable, or a bridge. The amount of weight required will depend on the depth and velocity of the water. Window-sash cord makes a

good line, though for deep, swift water No. 12 or 14 galvanized wire should be used below water since it offers less resistance to the current. Sometimes the weight and cable of the current meter are used.

Where the channel is permanent, accurate measurements are made before the velocity measurements, and an area curve drawn. For changing stations, the two measurements are taken at the same time. Particular care should be taken in making soundings in deep, swift water to insure that the line is vertical and that the depths are accurately measured, otherwise inaccurate and discordant discharges will result.

Velocity Measurements.—Discharge measurements may be made either at the gaging station or above or below it. The stream for some distance in each direction should be uniform in cross-section and fairly straight, and the bed and banks should be smooth and regular and free from vegetation. The velocity can be obtained in three ways: (1) by current meter; (2) by floats; and (3) by slope measurements.

Current-meter Method.—As illustrated in Fig. 55, the current meter is an instrument consisting essentially of a wheel which is revolved by the pressure of the flowing water and a device for indicating the number of revolutions of the wheel. In the type shown the axis of revolution is vertical. The instrument is used by lowering it into the water and noting the time required for a given number of revolutions. The time should not be less than 40 seconds. Each meter is accompanied by a rating table giving the velocity in feet per second corresponding to various numbers of revolutions for different time intervals. The indicating device gives either single revolutions for low velocities or every fifth revolution for high velocities by means of a loud click in a telephone receiver held to the ear or pinned to the coat of the observer. While any watch with a second hand can be used for obtaining the time, a stop watch is preferable.

In making velocity measurements by means of the current meter, the cross-section is divided into partial areas by perpendiculars at intervals of from 2 to 20 feet, as shown in Fig. 56, depending on the width of the stream and the regularity of the section; and velocity measurements are made in each vertical. The horizontal velocity of the filaments of water is not the same at all points in a stream, but differs with the depth of water, the roughness of the bed, and the distance from the bottom of the

stream and the surface of the water. The curves of equal velocity for a typical stream are shown in Fig. 57 and illustrate the effect of the above factors on the velocity. Thus, it is seen that if the meter is held at different depths, different velocities will

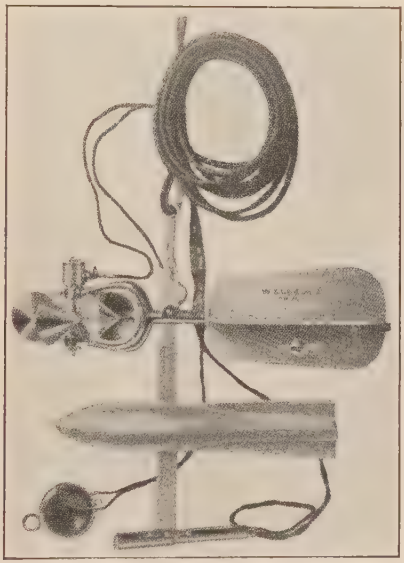


FIG. 55.—Price current meter.

be obtained. The average or mean velocity in each vertical or perpendicular boundary line is the value desired.

It has been found by numerous observations—and can be proved mathematically—that the velocity at 0.6 of the depth,

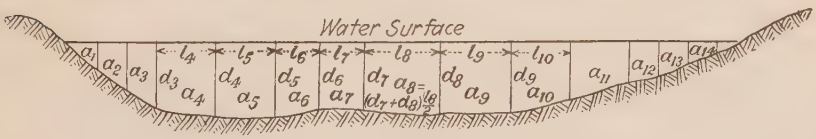
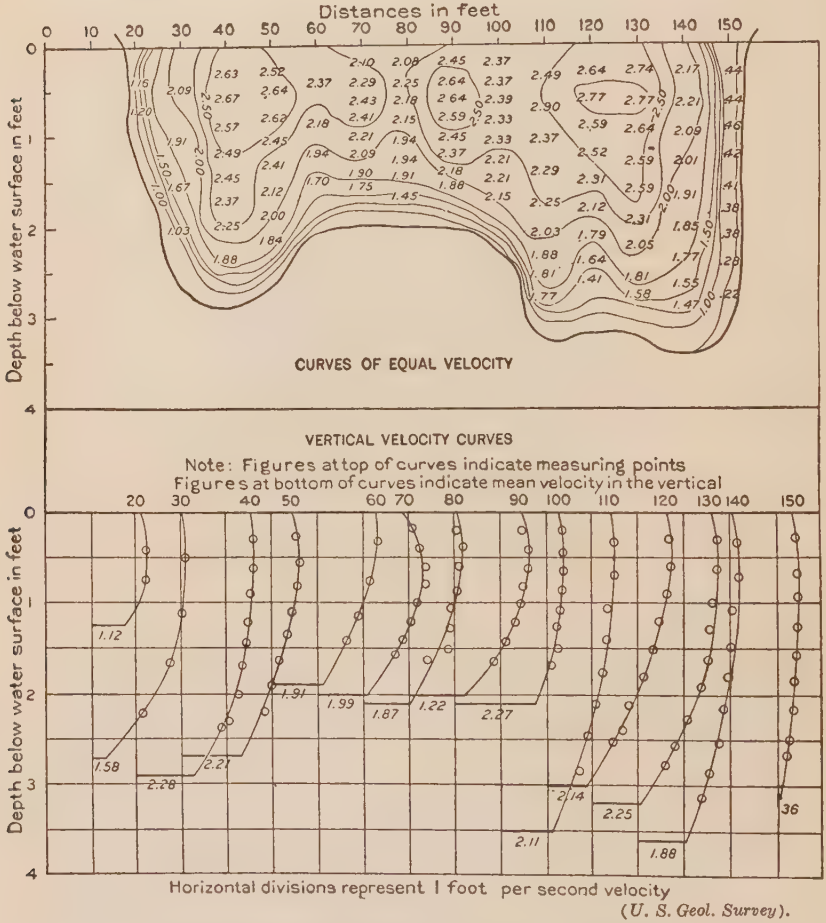


FIG. 56.—Sketch showing method of dividing channel cross-section into partial areas to determine discharge of stream.

(measured from the surface of the water,) is very close to the mean velocity in a vertical; and if time will permit only one measurement in each perpendicular, it should be made at that depth. Where the stream is either very deep or very shallow, however, this method should not be used.

(The average of the velocities obtained at 0.2 and 0.8 of the depth is more accurate than that obtained by the single reading at 0.6 depth, and this method is the one recommended by the United States Geological Survey for general stream gaging.)


Distances in feet
Depth below water surface in feet
CURVES OF EQUAL VELOCITY
VERTICAL VELOCITY CURVES
Note: Figures at top of curves indicate measuring points
Figures at bottom of curves indicate mean velocity in the vertical
Depth below water surface in feet
Horizontal divisions represent 1 foot per second velocity

(*U. S. Geol. Survey*).

FIG. 57.—Curves showing distribution of velocity in cross-section of typical stream.

The method used by the Agricultural Engineering Division of the Bureau of Public Roads, United States Department of Agriculture, is to take velocity measurements at the surface, at mid-depth, and as near the bottom as the meter can be lowered, and to obtain the mean velocity in the vertical by adding the

surface velocity, four times the velocity at mid-depth, and the bottom velocity, and dividing the sum by 6.

When the water is swift, the current meter will be carried downstream in spite of its weight, and as a result the measurement will not be made at the proper depth. To prevent this, the meter can be held in position by a guy wire attached to its upper end and passing over a pulley which in turn moves along a cable stretched across the stream above the meter section. In very swift streams, especially at flood periods, it is difficult to lower the meter to the

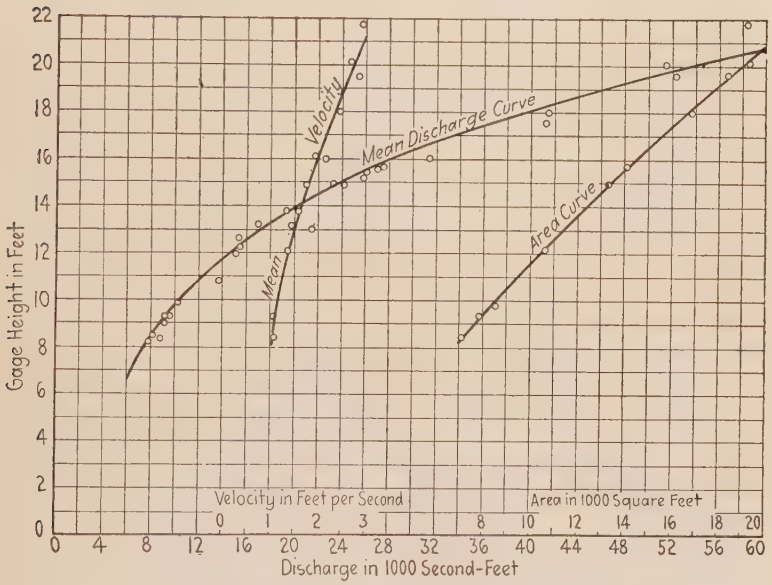


FIG. 58.—Rating curve for Illinois River at Peoria, Illinois.

required depth. In this case, the meter is held with the wheel from 0.5 to 1.0 foot beneath the surface. The velocity at this point is greater than the mean velocity in the vertical and must be multiplied by a coefficient, which for average conditions is about 0.90.

The discharge for each partial area is obtained by multiplying the average of the mean velocities in the two verticals bounding the area by the area in square feet. The sum of the discharges of the partial areas is the total discharge of the stream. This discharge divided by the cross-sectional area of the stream gives

Vm = Mean Velocity
D = Discharge ∴ Vm = D/A
A = Area

the *mean velocity* in the section. (It is to be noted that the mean velocity is a computed and not a measured quantity.) The mean velocities thus determined at varying stages are plotted as abscissas with gage heights as ordinates, and a mean-velocity curve drawn through the plotted points. A discharge curve is similarly constructed. Figure 58 shows the discharge, mean velocity, and area curves for the Illinois River at Peoria, Illinois.

The gage reading should be noted at the beginning of the meter measurements and observed at intervals during the measurements so that any change of stage may be recorded and proper allowance made in making the measurements and computing the discharges.

Float Method.—When the float method is used, a straight stretch of stream several hundred feet in length and of as uniform section as possible must be selected, and its ends marked either by range poles on the bank or by a line stretched across the stream. The area used in the computations of discharges is the average of the cross-sections at the ends of the course and at several intermediate points.

The velocity is obtained by noting the time required for floats to pass over the measured course. Surface floats, submerged floats, and tube floats are used for this purpose, though the first-named is the more common form. (A corked bottle weighted at the bottom and with a marker at the top makes an excellent surface float.) (Velocity measurements are made at varying distances from the bank, and the average taken as the mean surface velocity. This value is then multiplied by a coefficient, varying from 0.85 to 0.90, depending on the depth, and the product is considered the mean velocity of the stream, which when multiplied by the average cross-sectional area of the course gives the discharge.)

The submerged float is a weighted float attached by a cord to a floating buoy and adjusted so that it will float at the depth of mean velocity and give the mean velocity directly. The average depth along the course of the float is used in obtaining the depth of mean velocity.

The best form of tube float is a cylinder, 2 or 3 inches in diameter, graduated from the bottom up. Sufficient water is poured into the cylinder to make it float vertically and at the desired depth. The mean velocity is determined directly from the measured velocity of the float.

While the float method is less accurate than the current-meter method, it has the advantage of simplicity and cheapness. Also, during high water when the stream is full of drift and possibly floating ice, floats can be used where the current meter could be used only with the greatest difficulty.

Slope Method.—In this method the velocity is not measured but is computed from the Chezy formula, $V = c\sqrt{rs}$, in which V is the velocity, c a coefficient varying with the slope, hydraulic radius, and roughness of the banks and bed, r the hydraulic radius, and s the slope. The quantities measured are the slope of the water surface and the area of the cross-section of the channel, from which the hydraulic radius is computed. The roughness factor, n, required in computing the coefficient c is assumed, based on the judgment and experience of the observer. For the determination of slope, a straight and fairly uniform stretch of stream about ten times the width of the stream in length is required. The longer the course the greater will be the total fall, and the less the percentage of error in the measurements. The course chosen must be such that there are no abrupt changes in the slope of the bottom of the channel, and no obstructions such as logs, fallen trees, or bridge piers. The slope is obtained by taking the elevations of the water surface at a number of points on each side of the stream. These elevations are plotted on profile paper and a straight line drawn through their average positions. By using a large number of points in establishing the slope, errors in observation are largely eliminated and the result will be more accurate than if readings had been taken only at the ends of the course. While the elevations are being taken, the gage should be read frequently so that the elevations can be corrected for any change in stage which may occur while the measurements are being made. The time of taking each elevation should be recorded to the nearest minute.

If the slope course is to be used frequently, it may be desirable to establish gages at the ends of the course and at intermediate points. Such gages may consist of a vertical staff gage, graduated to hundredths of a foot, and protected by some form of stilling box to eliminate the effect of wave action. The zeros of the gages must be accurately referred to a permanent bench mark and checked frequently to note any changes which may have occurred.

The smaller the stream and the shorter the slope course, the more accurately will the elevations of the water surface have to be taken. For courses under 500 feet, the readings should be taken to thousandths of a foot. If the slope is to be measured at various stages, three or four semi-permanent slope posts should be established at each end of the course in the bed and bank at intervals of several feet and at right angles to the stream. These posts may be made of 2- by 4-inch studding firmly bedded in the ground and extending about 2.5 feet above the ground surface. By having a series of slope posts at different elevations, the elevation of the water can be determined at any stage. The tops of the posts should be notched and beveled in such a way as to provide a well-defined edge to measure from. The elevations of the edges should be carefully determined to the thousandths of a foot. In measuring from these slope posts to the surface of the water, a small measuring rod with a metal hook in its lower end and accurately graduated to hundredths of a foot should be used. The posts should be slightly inclined to facilitate the measurements. The measurements are taken by holding the hook gage vertically with the point of the hook at the water surface and the upper end of the gage against the beveled edge at the top of the slope post. The reading is estimated to thousandths of a foot, and, because of the undulations of the water surface, a series of ten readings should be quickly taken and the average used as the correct reading. A short length of stovepipe slipped over the post and into the water will still the wave action to some extent and make the readings more accurate.

The course is cross-sectioned at the ends and at as many intermediate points as conditions may require. Where the channel is fairly regular, uniform intervals of from 50 to 100 feet are satisfactory; where it is irregular, cross-sections must be taken at the points where the changes occur. The average of the areas thus obtained is multiplied by the velocity to obtain the discharge.

The hydraulic radius used in the velocity formula is the average of the hydraulic radii computed for the several cross-sections. The sections are plotted to a scale of from 5 to 20 feet to the inch, the areas are obtained with a planimeter, and the wetted perimeters by a little instrument known as a "map measurer," or by stepping off the distances with dividers.

The coefficient c in the velocity formula is computed from Kutter's formula or taken from a table or diagram derived from it. Table XVI gives values of c for different values of r, s, and n.

The roughness factor, n, allows for all the conditions which retard the flow of the water, and includes not only the friction between the flowing water and the bed and banks of the stream, but also the effect of vegetation, trees, logs, stumps, and drift in the channel as well as non-uniformity in the shape and size of the channel. A complete discussion of the roughness factor together with values to be used for different conditions is given in Chapter V. The choice of a proper value of n is probably the most important and most difficult decision which the engineer has to make in using the slope method for determining discharge.

The slope method is less accurate than the other methods described and its use is limited almost entirely to flood flows where the other methods are more or less impracticable. It is also used in determining the discharge after the flood has subsided, the slope of the high-water surface being obtained by measuring the elevations of high-water marks left by the flood.

When a stream is out of its banks, the discharge in and over the channel must be computed separately from that in the flood plain. The flood plain is usually rougher than the channel and offers greater resistance to the flow of the water, hence a different roughness factor must be used for this portion.

Mean-velocity Curves.—The total discharge is divided by the area of the measured section to determine the mean velocity. The mean velocities thus obtained at different stages of the stream are plotted as abscissas against gage heights as ordinates. The curve drawn through these points is the mean-velocity curve for the station. Above low water at least, the mean-velocity curve approximates a parabola with axis vertical and its curvature concave to the Y-axis. For the higher stages the curve approaches a straight line. The curve always intersects the Y-axis at the stage of zero flow.

Where discharge measurements have been made up to flood stage, the only advantage in constructing a mean-velocity curve is that it provides a check on the accuracy of the measurements. Where the measurements do not include the higher stages, the curve can be extended tentatively either as a tangent from

the last plotted value or as a hyperbola, and used in extending the discharge curve to the flood stage.

Rating Curve.—The station rating curve is drawn through the plotted values of discharge, similarly to the area and mean-velocity curves. These three curves are illustrated in Fig. 58.

The curves are usually drawn on ordinary cross-section paper, although logarithmic cross-section paper may be used. The rating curve is convex to the Y-axis and cuts that axis at the point of zero flow, which may be at, or above, the bottom of the stream. The curve is not a mathematical curve, but is usually made up of a series of parabolas. The advantage of using logarithmic paper is that the curves become straight lines and hence can be extended more readily.

The chief difficulty in constructing a satisfactory rating curve is that the flow of a stream is seldom uniform, but is constantly changing; and, as previously mentioned, the discharge of a rising stream is not the same as that of a falling one even for the same stage, because of the change in the slope. (The slope of a stream at a given station depends not upon the actual rise and fall of the stream, but upon *the rate of rise and fall.*) Hence, discharge measurements made on a rising stream will not only differ from those made on a falling one at the same stage, but also measurements on a rising or a falling stream will themselves vary depending upon the rate of rise and fall. A large number of observations at the same stage, therefore, give discordant results, and the discharges when plotted for all stages will define an area rather than a line. The mean discharge curve will fall within this area. For average streams, however, these differences occur mainly at high stages and the lower portion of the curve is fairly well defined.

Rating Table.—The discharges for various gage heights, differing by hundredths, half-tenths, or tenths, are scaled from the rating curve and tabulated. The readings taken from the curve are first adjusted so that the differences in discharge for successive stages will increase or decrease gradually and regularly. The rating table for the discharge curve in Fig. 58 is given as Table XIV. A discharge curve and rating table can be used only as long as the sections at the gaging station and at the point of control remain unchanged.

After a satisfactory rating table has been prepared, the only observation necessary is the reading of the gage daily—sometimes

more often—and taking the corresponding discharge from the table.

TABLE XIV.—RATING TABLE FOR THE ILLINOIS RIVER AT PEORIA, ILLINOIS, 1903 TO 1906

Gage height	Discharge	Gage height	Discharge	Gage height	Discharge
6.60	6,100	8.50	8,140	10.80	11,960
6.70	6,200	8.60	8,260	11.00	12,390
6.80	6,300	8.70	8,390	11.20	12,830
6.90	6,400	8.80	8,520	11.40	13,280
7.00	6,500	8.90	8,650	11.60	13,740
7.10	6,600	9.00	8,790	11.80	14,220
7.20	6,700	9.10	8,930	12.00	14,700
7.30	6,800	9.20	9,080	12.20	15,210
7.40	6,900	9.30	9,230	12.40	15,730
7.50	7,000	9.40	9,390	12.60	16,270
7.60	7,110	9.50	9,550	12.80	16,820
7.70	7,220	9.60	9,710	13.00	17,400
7.80	7,330	9.70	9,870	13.20	17,990
7.90	7,440	9.80	10,040	13.40	18,590
8.00	7,550	9.90	10,210	13.60	19,210
8.10	7,660	10.00	10,390	13.80	19,840
8.20	7,780	10.20	10,760	14.00	20,480
8.30	7,900	10.40	11,140	15.00	24,090
8.40	8,020	10.60	11,540	16.00	28,600

Weir Stations.—The discharge of a stream can be obtained also by noting the head of water on the crest of an existing dam. Such a dam forms a broad-crested weir with a free overfall—except possibly during extreme high water—and the flow over it conforms to definite hydraulic laws. If the dam was constructed for power or navigation purposes, the diversion of water around it must also be measured and added to the discharge over the crest.

To give satisfactory results, the dam must be tight, must confine the flow at high stages, and its crest must be level, free from flashboards, of a type for which coefficients have been determined experimentally, and of sufficient height to give a free overfall at high water. Also, if there is diversion of water around the dam, it must be capable of being readily measured. The flow

over such a dam can be computed from the fundamental weir formula,

$$Q = \tfrac{2}{3} cl\sqrt{2g}\, H^{3/2} \qquad (1)$$

in which c is a coefficient varying with the width and shape of the crest, l the length of the dam, g the acceleration of gravity, and H the head of water on the crest. All of these factors are measured with the exception of c, which must be determined experimentally.

In *Water Supply Paper* 200 of the United States Geological Survey, R. E. Horton summarizes the results of the experiments made to determine the coefficient c for broad-crested weirs, and shows that the formula

$$Q = 2.64 l H^{3/2} \qquad (2)$$

may be safely used for computing the discharge over dams whose crests are 2 feet and over in width when the head of water is

TABLE XV.—DISCHARGE IN SECOND-FEET PER FOOT OF CREST LENGTH FOR CERTAIN BROAD-CRESTED WEIRS

Formula: $Q = 2.64 l H^{3/2}$

Head H in feet	0	1	2	3	4	5	6	7	8	9	10
0.00	0.000	2.64	7.47	13.7	21.1	29.5	38.8	48.9	59.7	71.3	83.5
0.05	0.030	2.84	7.75	14.1	21.5	30.0	39.3	49.4	60.3	71.9	84.1
0.10	0.083	3.04	8.03	14.4	21.9	30.4	39.8	49.9	60.8	72.5	84.7
0.15	0.153	3.26	8.32	14.8	22.3	30.8	40.3	50.5	61.4	73.1	85.4
0.20	0.236	3.47	8.61	15.1	22.7	31.3	40.8	51.0	62.0	73.7	86.0
0.25	0.330	3.69	8.91	16.5	23.1	31.8	41.2	51.5	62.6	74.3	86.6
0.30	0.434	3.91	9.21	15.8	23.5	32.2	41.7	52.1	63.1	74.9	87.3
0.35	0.547	4.14	9.51	16.2	24.0	32.7	42.2	52.6	63.7	75.5	87.9
0.40	0.668	4.37	9.82	16.6	24.4	33.1	42.7	53.1	64.3	76.1	88.5
0.45	0.797	4.61	10.1	16.9	24.8	33.6	43.2	53.7	64.8	76.7	89.2
0.50	0.934	4.85	10.4	17.3	25.2	34.0	43.7	54.2	65.4	77.3	89.8
0.55	1.08	5.09	10.8	17.7	25.6	34.5	44.3	54.8	66.0	77.9	90.5
0.60	1.23	5.34	11.1	18.0	26.0	35.0	44.8	55.3	66.6	78.5	91.1
0.65	1.38	5.60	11.4	18.4	26.5	35.4	45.3	55.9	67.2	79.1	91.8
0.70	1.55	5.85	11.7	18.8	26.9	35.9	45.8	56.4	67.7	79.8	92.4
0.75	1.71	6.11	12.0	19.2	27.3	36.4	46.3	57.0	68.3	80.4	93.0
0.80	1.89	6.38	12.4	19.6	27.8	36.9	46.8	57.5	68.9	81.0	93.7
0.85	2.07	6.64	12.7	19.9	28.2	37.4	47.3	58.1	69.5	81.6	94.4
0.90	2.25	6.91	13.0	20.3	28.6	37.8	47.8	58.6	70.1	82.2	95.0
0.95	2.44	7.19	13.4	20.7	29.1	38.3	48.4	59.2	70.7	82.9	95.6
1.00	2.64	7.47	13.7	21.1	29.5	38.8	48.9	59.7	71.3	83.5	96.3

such that the nape does not adhere to the downstream face of the dam for low heads nor become detached from the crest for high heads. The latter condition is likely to exist when the head becomes from one and a half to two times the width of the crest. Table XV gives discharges per foot of length of dam for heads up to 10 feet.

The head of water on the dam is obtained by subtracting the elevation of the crest from the elevation of the surface of the water a sufficient distance above the dam to include the head due to the velocity of approach. The latter elevation should be read to hundredths of a foot by means of a hook gage. If the crest of the dam is not level throughout its length, the stream must be divided into sections and the discharge through each determined separately.

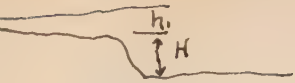
The diversion of water around the dam may be so intermittent as to make it difficult and expensive to determine the amount of water diverted. In this case, the weir method cannot be used satisfactorily.

Contracted-opening Method.—Another method of determining the discharge of a stream when it is out of its banks is to measure the cross-sectional area at some restricted opening in the channel and to compute the velocity of flow through it. A bridge with approach embankments over the flood plain provides such an opening, since the embankments pond the water above and cause all the discharge to pass through the bridge. As the inflow of water above the bridge is greater during flood periods than the discharge through the bridge, the water will back up and the elevation of the surface of the water above the bridge will be considerably higher than that below, and the two surfaces will be connected by a sudden drop-off curve in the surface of the water just above the entrance to the opening. The height of the drop-off curve will, of course, depend on the ratio of the inflow to the outflow. As the water moves through the drop-off section, the loss of head is accompanied by an increase in the velocity, and the maximum velocity will obtain at the point of maximum contraction, which is the point at which the velocity is computed and the area measured. The velocity is computed from the formula,

$$V = \sqrt{2gh} \tag{3}$$

in which h is the velocity head at the point of maximum contraction. The velocity head is computed from Bernoulli's formula,

$$h = H + h_1 - h_2 \tag{4}$$



in which H is the height of the drop-off curve in feet, h_1 the head due to the velocity of approach, and h_2 the friction head lost in the drop-off section. H is obtained by: (1) taking a sufficient number of elevations of the surface of the water above the drop-off section to define the slope of that surface when the elevations are plotted; (2) taking elevations of the water surface through and below the contracted section to define the slope of that surface; (3) extending the two plotted slopes to the point of maximum contraction; and (4) noting the difference in the elevations of the two surfaces at that point, which is the desired quantity. If the measurements are made after the flood has subsided, reliable high-water marks must be used in determining the two slopes.

The velocity of approach, v_1, is usually determined by assuming successive values until one is obtained which, when multiplied by the area of the section above the drop-off point, gives the same discharge as that obtained at the contracted opening. The velocity head, h_1, corresponding to this velocity is computed from the formula, $h_1 = \frac{{v_1}^2}{2g}$.

The friction head is computed from the approximate formula,

$$h_2 = ls\,\frac{{v_1}^2 + v^2}{2{v_1}^2} \tag{5}$$

in which l is the sum of the lengths of the drop-off and contracted sections, s the surface slope above the drop-off section, v_1 the velocity of approach, and v the velocity at the point of maximum contraction. In deriving this formula it is assumed that the average velocity in the drop-off section is represented more accurately by the term, $\sqrt{\frac{{v_1}^2 + v^2}{2}}$, than by $\frac{1}{2}(v_1 + v)$. Since v cannot be found until h_2 is known, successive values of h_2 must be assumed until a value is found which satisfies formula (5).

In measuring the area of the cross-section at the point of maximum contraction to be used in determining the discharge, only the effective area should be used, that is, all holes which have been scoured in the bottom of the channel in which the water simply moves around in eddies should not be included in the cross-section. Also, a contraction coefficient should be applied to the area to allow for the loss of cross-section caused by the contractions at the abutments and piers. This coefficient will in general be not less than 0.90 or 0.95.

Where the drop in surface is several feet, the head due to velocity of approach and the friction head will be comparatively small, and fairly accurate discharge determinations can be obtained.

The contracted-opening method was used by the engineers of the Miami Conservancy District in estimating the discharge of the 1913 flood, and the results obtained were in most instances more accurate than those obtained by the slope-area method. A complete description of the theory and application of this method is given in *Technical Report*, Part IV, of the Miami Conservancy District.

CHAPTER V

FLOW OF WATER IN OPEN CHANNELS

In designing ditches and in computing the discharge of existing channels, both natural and artificial, the engineer must be familiar with the laws which govern the velocity of flowing water, and must understand the use and limitations of the empirical formulas which attempt to express those laws.

In this chapter, the gradual development of velocity formulas is given briefly, and the formulas in general use are explained, discussed, and compared.

Surface Slope.—The motive force of flowing water is due entirely to the surface slope. If the water surface is level, there can be no flow; but if it is higher at one point than at another, the water will fall or flow from the higher to the lower point by virtue of gravity. If it were not for the resistances caused by friction, the water would acquire a greater velocity as it moved along, in accordance with the law of falling bodies, and the velocity at every point in a given cross-section would be the same. Even the smoothest channels, however, cause friction, and the greater the velocity the greater is the effect of friction. When the same quantity of water passes a given cross-section in equal successive intervals of time—that is, when steady flow exists—the mean velocity of the water is such that the accelerating force caused by the surface slope is just balanced by the retarding force of friction, and the water moves at a constant mean velocity as long as the channel conditions remain unchanged.

The friction between the moving water and the bottom and sides of a channel causes an unequal distribution of velocity as illustrated in Fig. 57, the maximum velocity occurring at the point most distant from the surfaces causing friction. Due to surface tension and atmosphere, there is a resistance to flow at the surface as well as at the bottom and sides, and hence the point of maximum velocity is over the deepest part of the channel and at a distance below the surface, depending upon the depth. For shallow channels, it is near the surface; for deep ones, it

is about one-third the depth below the surface. The method of determining the mean velocity from current meter measurements is described in Chapter IV, page 92. In designing channels, a slope formula is used in which the velocity computed is the mean velocity in the cross-section.

As the surface slope increases, the velocity also increases although at a smaller rate. (Other factors remaining constant, the mean velocity varies as the square root of the slope.)

Under conditions of steady flow and for straight channels of uniform section and slope, the water surface is parallel to the bed of the channel. All velocity formulas are based on the assumption of steady flow.

Hydraulic Radius.—The velocity varies also with the hydraulic radius or hydraulic mean depth. (The hydraulic radius is equal to the area of the channel section divided by the wetted perimeter.) For channels of the same area and slope, the one having the smallest wetted perimeter will have the largest hydraulic radius and the highest velocity, (since the retarding effect of friction is proportional to the length of the surfaces in contact.) For example, the velocity in a rectangular channel 20 feet wide and 10 feet deep is about 60 per cent greater than in a channel 100 feet wide and 2 feet deep, for the same slope and the same degree of roughness of channel. The area is the same in each case, but the wetted perimeter of the former channel is only 40 feet, while in the latter it is 104 feet. (Other factors remaining constant, the velocity varies as the one-half to the two-thirds power of the hydraulic radius.)

Channel Resistances.—The resistances which retard the flow of water in open channels are many and varied. The principal sources of resistance are the (material forming the channel) the irregularity of the wetted perimeter, changes in size and shape of cross-section, curvature, weeds and growths of various kinds, scouring and silting, and obstructions such as logs, snags, sand bars, and bridge piers and abutments. It is impossible to assign definite values to each of these causes of retardation of flow. In practice, the resultant of all these forces is expressed by a coefficient of roughness determined experimentally for channels of different types and conditions. Thus, it is evident that the flow of water, especially in channels of earth, presents a complicated problem and one which cannot be solved with mathematical exactness.

Discharge.—The general formula for the discharge of a stream is:

$$Q = AV \tag{6}$$

where Q is the discharge in cubic feet per second; A, the area of the channel cross-section in square feet; and V, the mean velocity in feet per second. Where the velocity is measured directly by the current meter, A is the area at the meter section; but where the velocity is measured by some form of float or is computed by a slope formula, A is the average area of the float or slope section.

Early Velocity Formulas.—When the importance of the subject is considered and the fact that from the earliest times artificial channels were constructed for irrigation purposes, and aqueducts for supplying cities with their water supply, it is a source of wonder that the laws of flowing water were so slow in being developed and that our modern formulas are not more definite than they are. Even the Romans, who built 250 miles of aqueducts for supplying the city of Rome with water, had no conception of velocity as a factor in the flow of water, and measured the discharge by the diameter of the outlet pipe. In fact it was not until the fourteenth century that hydraulics was regarded as a science.

The first real scientific investigators were Castelli and Torricelli, pupils of Galileo. It was Castelli, who in 1628 introduced the velocity factor as an element in the discharge of a stream; but his conclusion was that the velocity varied directly as the head. Toricelli, in 1643, went a step further and declared that flowing water followed the law of falling bodies, and that the velocity varies as the square root of the head.

In 1738, the mathematician Bernoulli introduced the gravity factor and gave us the fundamental hydraulic formula $V = \sqrt{2gh}$. Fifteen years later, Brahms pointed out that the velocity did not accelerate in accordance with the law of gravity, because of frictional resistances offered by the bed and sides of the channel; and he assumed that the resistance was proportional to the hydraulic radius. This theory of Brahms was expressed algebraically by Chezy in 1775 in the formula,

$$V = c\sqrt{rs} \tag{7}$$

in which s is the slope; r, the hydraulic radius; and c, a varying coefficient embracing all the retarding influences other than r and s.

Although the Chezy formula was the first practical formula ever proposed for the uniform flow of water in open channels, it is the basis of all modern velocity formulas. Chezy did not give the values of c to be used in his formula, but subsequent investigators have proposed various values.[1] Thus, Young adopted 84.3 for large streams; Eytelwein, 93.4; D'Aubisson, 95.6 for velocities over 2 feet per second; Downings and Taylor, 100 for large and swift streams; Leslie, 68 for small streams and 100 for large ones; Beardmore, 94.2; and Stevenson, 69 for small streams and 96 for large streams.

Dubuat in 1786 established the principles that the motive force of flowing water is due entirely to the surface slope, and the resistances to the viscosity of the water and to the friction upon the bed. Starting with the law that the forces which keep uniformly flowing water in motion are equal to the sum of all the resistances, he adopted the rather complex formula,

$$V = \frac{88.49(r^{\frac{1}{2}} - 0.03)}{\left(\frac{1}{s}\right)^{\frac{1}{2}} - L\left(\frac{1}{s} + 1.6\right)^{\frac{1}{2}}} - 0.086(r^{\frac{1}{2}} - 0.03) \qquad (8)$$

in which L is the hyperbolic logarithm and is equal to the common logarithm multiplied by 2.302585.

In 1800, Coulomb published a paper in which he showed that the channel resistances could be represented by a function consisting of two terms, one containing the first and the other the second power of the velocity. Coulomb's theory was used by Girard in 1803 in developing the formula,

$$V = (2.69 + 26{,}384rs)^{\frac{1}{2}} - 1.64 \qquad (9)$$

De Prony, in 1804, proposed a formula of the same type but with different constants. His formula for canals was,

$$V = (0.0556 + 10{,}593rs)^{\frac{1}{2}} - 0.2357 \qquad (10)$$

In 1814, Eytelwein deduced a simple formula which was widely used for a time, and is still used in a modified form. It is,

$$V = 0.9091(2Hr)^{\frac{1}{2}} \qquad (11)$$

in which H is the fall in one English mile. This formula can be reduced to the form of the Chezy formula, and becomes

$$V = 93.4\sqrt{rs} \qquad (12)$$

[1] HUMPHREYS and ABBOT, "Physics and Hydraulics of the Mississippi River," pp. 214–215.

Another formula which is frequently referred to is that of Weisbach which was published in 1846. It has the same form as the de Prony formula but with different coefficients. The Weisbach formula is

$$V = (0.00024 + 8675rs)^{\frac{1}{2}} - 0.0154 \qquad (13)$$

Other investigators who contributed to the development of velocity formulas were Lombardini (1844), Dupuit (1848), Ellet (1851), Stevenson (1858), Humphreys and Abbot (1861), and Darcy and Bazin (1865).

The formulas which were proposed prior to 1869 are valuable only in so far as they contributed to the development of modern formulas. The formulas which are now used by engineers will be discussed in more detail.

Kutter's Formula.—The best-known and most widely used formula for computing the flow of water in open channels is that derived in 1869 by E. Ganguillet and W. R. Kutter, two Swiss Government engineers, and generally referred to as Kutter's formula. The general form of the Chezy formula was accepted as a basis, and the work of these investigators was to determine an expression for the coefficient c which would contain the several factors combined in that term, and which could be more readily used. This formula expressed in metric units which were used in its development is,

$$V = \frac{23 + \frac{1}{n} + \frac{0.00155}{s}}{1 + \left(23 + \frac{0.00155}{s}\right)\frac{n}{\sqrt{r}}}\sqrt{rs} \qquad (14)$$

Expressed in English units, the formula becomes

$$V = \left[\frac{41.6 + \frac{1.811}{n} + \frac{0.00281}{s}}{1 + \left(41.6 + \frac{0.00281}{s}\right)\frac{n}{\sqrt{r}}}\right]\sqrt{rs} \qquad (15)$$

in which,

V = the mean velocity in feet per second.
s = the slope, or fall per foot.
r = the hydraulic radius, or area of cross-section divided by the wetted perimeter.
n = a coefficient depending upon all the characteristics of a channel which cause retardation of flow, such as roughness of material in bed and sides, irregularity in bed and sides, irregularity in cross-section and profile, vegetation, obstructions, etc.

TABLE XVI.—VALUES OF c FROM KUTTER'S FORMULA FOR USE IN THE CHEZY FORMULA, $V = c\sqrt{rs}$

Slope	r, feet	n = Coefficient of roughness									
		0.013	0.015	0.017	0.020	0.025	0.030	0.035	0.040	0.050	0.060
s = 0.00005 = 1 in 20,000 = 0.264 feet per mile	1	104	89	77	64	49	40	34	29	23	19
	2	124	107	94	79	62	51	44	38	30	25
	3	136	118	104	88	71	59	50	44	35	29
	4	145	126	111	95	77	64	56	49	39	33
	6	156	137	122	105	85	72	63	56	45	38
	8	163	144	129	111	91	78	68	61	49	42
	10	169	149	134	116	96	82	72	64	53	45
	12	173	153	138	120	99	86	75	68	56	48
	16	180	160	144	126	106	91	81	73	60	52
	20	184	165	149	131	110	96	85	77	64	55
	30	192	172	157	139	118	103	92	84	71	62
	50	201	181	165	148	127	112	101	93	79	70
	100	210	190	175	158	137	123	112	104	90	81
s = 0.0001 = 1 in 10,000 = 0.528 feet per mile	1	109	93	81	67	52	42	35	31	24	19
	2	127	109	96	81	64	53	45	39	31	25
	3	137	119	104	89	71	59	51	45	35	29
	4	143	125	111	94	76	64	55	49	39	32
	6	152	134	119	102	84	70	61	54	44	37
	8	158	139	124	107	88	75	66	59	47	40
	10	162	143	128	111	92	78	69	62	50	43
	15	169	150	135	118	98	85	75	68	56	48
	20	173	154	139	122	102	89	79	71	60	52
	30	179	160	145	128	108	94	84	77	65	57
	50	185	166	151	134	114	100	91	83	71	63
	100	191	172	158	140	121	108	98	91	79	70
s = 0.0002 = 1 in 5,000 = 1.056 feet per mile	1	113	96	83	69	54	44	37	32	24	20
	2	129	111	97	82	64	54	45	40	31	26
	3	137	119	105	89	72	59	51	45	35	29
	4	143	125	111	94	76	63	55	48	38	32
	6	150	132	117	100	82	69	60	53	43	36
	8	155	137	122	105	87	73	64	57	46	39
	10	158	140	125	108	89	76	67	60	49	42
	15	164	145	131	113	95	82	72	65	53	46
	20	168	149	134	117	98	85	76	68	57	49
	30	172	154	139	122	103	89	80	73	61	53
	50	177	158	143	126	108	94	85	78	66	58
	100	182	163	148	131	113	99	90	83	72	64
s = 0.0004 = 1 in 2,500 = 2.112 feet per mile	1	115	98	85	70	55	44	37	32	25	20
	2	130	112	98	83	65	54	45	40	31	26
	3	138	119	105	89	71	59	51	45	35	29
	4	142	124	110	94	76	63	55	48	38	32
	6	149	130	116	99	81	68	60	53	43	36
	10	157	138	123	107	88	75	66	59	48	41
	20	164	146	131	115	96	82	73	66	55	48
	50	173	154	139	123	104	91	82	75	63	56
	100	177	158	143	127	108	95	87	80	68	61
s = 0.001 = 1 in 1,000 = 5.28 feet per mile	1	116	99	86	71	56	45	38	33	25	20
	2	130	112	98	83	66	54	46	40	31	26
	3	138	119	105	89	71	59	51	45	35	29
	4	142	124	110	93	75	63	54	48	38	32
	6	149	130	116	99	81	68	59	52	42	36
	10	155	136	122	105	87	74	65	58	47	40
	20	163	144	129	113	94	81	72	65	54	47
	50	170	151	137	120	101	89	79	72	61	54
	100	174	155	141	124	105	94	85	77	66	59
s = 0.01 = 1 in 100 = 52.8 feet per mile	1	117	99	87	72	56	45	38	33	25	21
	2	130	112	99	83	66	55	46	40	31	26
	3	138	119	105	89	71	59	51	45	35	29
	4	142	123	109	93	76	63	55	48	38	32
	6	148	129	115	99	81	68	59	52	42	36
	10	154	136	121	105	86	73	65	58	47	40
	20	161	143	128	112	93	80	71	64	53	46
	50	168	150	135	119	100	87	78	71	60	53
	100	172	153	139	122	104	91	82	75	64	58

This formula differs from all others in that the coefficient c contains a slope factor, which was introduced as the result of the study of the measurements made between 1850 and 1860 by Humphreys and Abbot on the Mississippi River, and those made by Bazin on European streams. Although this formula is rather complex, its use has been facilitated by tables and diagrams from which values of c for various values of r, s, and n are quickly obtained. Table XVI, reproduced with some modifications from Trautwine's "Engineers' Pocket Book," is such a table.

Relation between c, r, s, **and** n.—In order that the relations between c, r, s, and n may be more readily grasped, the curves

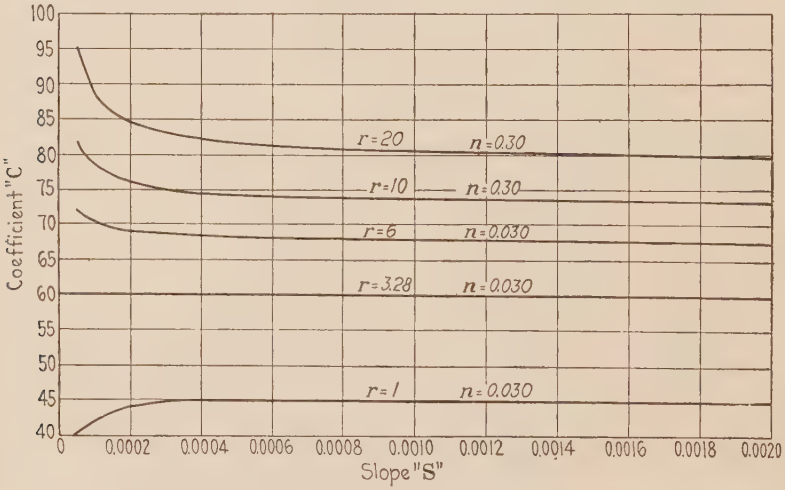


Fig. 59.—Relation between slope and coefficient.

in Figs. 59, 60, and 61 have been prepared. The general relations shown by these curves can be stated as follows:

1. The coefficient c is independent of the slope when the hydraulic radius is 3.28 feet, and is equal to $\frac{1.811}{n}$. When the hydraulic radius is greater than 3.28 feet, c decreases as the slope increases. However, c is practically constant for all values of s greater than 0.0003 (Fig. 59).

2. The coefficient c increases as the hydraulic radius increases, but the rate of increase in c becomes less as the hydraulic radius becomes larger. Also, as the hydraulic radius increases, the coefficient c is less affected by a change in the roughness factor n (Fig. 60).

3. The coefficient c decreases as the roughness factor n increases, but the rate of decrease becomes less as the roughness factor becomes greater (Fig. 61).

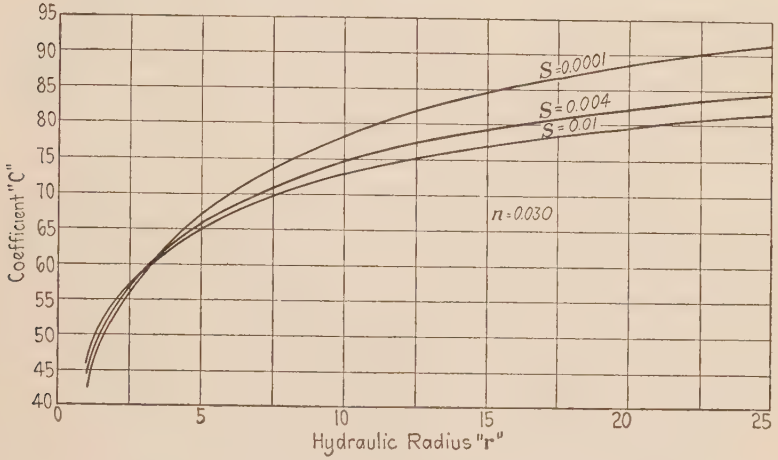


FIG. 60.—Relation between hydraulic radius and coefficient.

Kutter's formula has been criticized by some because it contains a slope factor in the coefficient c, and by others because c decreases as s increases when r is over 3.28 feet. Measurements,

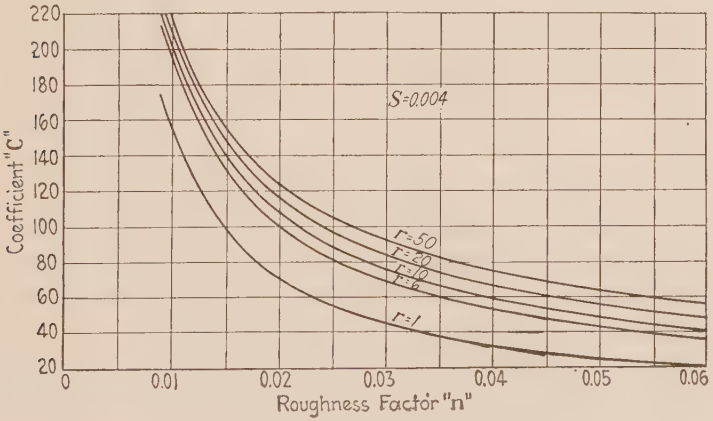


FIG. 61.—Relation between roughness factor and coefficient.

however, made on the Mississippi River at Carollton by the Mississippi River Commission (since Humphreys and Abbot's investigation), on the Bogue Phalia River in Mississippi by the

Morgan Engineering Company, on the Irrawaddy in Burmah, and on the Volga and other European streams, seem to show that for large channels with flat slopes c does decrease with an increase in s. For small channels, this relation is not so evident, and it is possible that for such channels the effect of s on c is negligible. Since the slopes used in designing channels are ordinarily larger than 0.0003, the slope term in Kutter's formula has had little effect on the results obtained from its use, and from a practical standpoint this term could be omitted.

The Roughness Coefficient.—In using Kutter's formula in any given case, r and s are known, but n is assumed; and it is in the assumption of a value of n that the greatest difficulty arises. The authors of this formula determined the value of n for a large

TABLE XVII.—HORTON'S VALUES OF n FOR KUTTER'S AND MANNING'S FORMULAS

Surface	Perfect	Good	Fair	Bad
Canals and ditches:				
Earth, straight and uniform	0.017	0.020	0.0225[1]	0.025
Rock cuts, smooth and uniform	0.025	0.030	0.033[1]	0.035
Rock cuts, jagged and irregular	0.035	0.040	0.045	
Winding sluggish canals	0.0225	0.025[1]	0.0275	0.030
Dredged earth channels	0.025	0.0275[1]	0.030	0.033
Canals with rough, stony beds, weeds on earth banks	0.025	0.030	0.035[1]	0.040
Earth bottom, rubble sides	0.028	0.030[1]	0.033[1]	0.035
Natural stream channels:				
(1) Clean, straight bank, full stage, no rifts or deep pools	0.025	0.0275	0.030	0.033
(2) Same as (1), but some weeds and stones	0.030	0.033	0.035	0.040
(3) Winding, some pools and shoals, clean	0.033	0.035	0.040	0.045
(4) Same as (3), lower stages, more ineffective slopes and sections	0.040	0.045	0.050	0.055
(5) Same as (3), some weeds and stones	0.035	0.040	0.045	0.050
(6) Same as (4), stony sections	0.045	0.050	0.055	0.060
(7) Sluggish river reaches, rather weedy or with very deep pools	0.050	0.060	0.070	0.080
(8) Very weedy reaches	0.075	0.100	0.125	0.150

[1] Values commonly used in designing.

number of streams of different characteristics by measuring the discharge and computing n from the formula. From these measurements, values of n for different degrees of roughness were tabulated. From time to time, as the results of experiments, other tables have been proposed giving more detailed information in regard to n. Probably the best of these is that proposed in 1916 by R. E. Horton[1] and given in part as Table XVII. In practice, however, all such tables have to be modified to take into account the characteristics peculiar to a given stream. As previously stated, n is dependent upon: (1) the material forming the bed and sides; (2) vegetation or other obstructions in the channel; (3) the irregularity of the wetted perimeter and the cross-section; (4) curvature or bends in channel; (5) stage of stream; and (6) silting and scouring. While the first two items mentioned are the most important, the others cannot be ignored.

Material of Bed and Banks.—The nature of the material forming the wetted perimeter is of primary importance. In alluvial streams where the material is sand, clay, loam, or silt, the retarding effect is much less than it is where the material is coarse gravel or boulders. In the first instance, changes of stage do not affect the value of n to any appreciable extent; in the second instance, the roughness coefficient varies considerably with the stage, since the larger boulders collect on the bed of the stream and make it rougher than the banks, and during low stages the wetted perimeter, as a whole, is rougher than at high stages. On the other hand, at high stages, a portion of the energy of the stream is used in rolling the boulders downstream, thus increasing the coefficient.

Vegetation and Other Obstructions.—This factor is important especially in small channels. The effect of vegetation on the roughness coefficient can be seen from the following examples:

1. Measurements made on Old Town Creek,[2] near Tupelo, Mississippi, in 1913 and 1914 before and after the channel had been cleared of vegetation, brush, and other obstructions to flow, gave values of n of 0.049 and 0.0332, respectively, for a hydraulic radius of 12.7 feet; and 0.055 and 0.0355, respectively, for a hydraulic radius of 5.4 feet. Views of this channel before and

[1] HORTON, ROBERT E., "Some Better Kutter's Formula Coefficients," *Eng. News*, Feb. 24, 1916.

[2] Bull. 832, Bureau of Public Roads, United States Department of Agriculture, June, 1920.

after clearing are given in Figs. 62 and 63, and the hydraulic elements of the channel are given in Table XVIII.

FIG. 62.—View of channel of Old Town Creek, near Tupelo, Miss., before clearing.

2. Measurements made by the engineers of the Miami Conservancy District[1] on the Miami and Erie Canal at Dayton,

FIG. 63.—View of channel of Old Town Creek, near Tupelo, Miss., after clearing.

Ohio, on Aug. 27, 1914, when large bunches of weeds and grass covered the banks—although the vegetation had been partially

[1] *Technical Report*, Part IV, Miami Conservancy District.

TABLE XVIII.—RESULTS OF EXPERIMENTS MADE IN LEE COUNTY, MISSISSIPPI

Old Town Creek dredged channel near Tupelo, Mississippi (before clearing), 1913

No.	Date of observation	Average maximum depth, feet	Average surface width, feet	Discharge, second-feet	Average cross-section, square feet	Mean velocity, feet per second	Mean hydraulic radius, feet	Slope of water surface	Coefficient in formula $V=C\sqrt{RS}$ C	Coefficient of roughness n	Description of channel
1	Mar. 4, 1913	5.4	22.9	95.3	76.2	1.25	2.90	0.000518	32.2	0.0550	*Course*, straight; 1,224 feet long. *Cross-section*, slight and gradual variations in shape. *Side slopes*, right side irregular, left side very irregular. *Bottom*, irregular and uneven. *Soil*, varies from black waxy clay at top to yellow clay at bottom. *Condition*, right side slope and part of bottom practically covered with small saplings, brush, and cane; occasional growths of vegetation on other side slope. *Constructed*, December, 1907.
2	Mar. 3, 1913	5.8	22.0	125.7	85.5	1.47	3.12	0.000513	36.8	0.0490	
3	Mar. 24, 1913	6.7	25.9	179.6	108.2	1.66	3.55	0.000444	41.8	0.0450	
4	Mar. 21, 1913	10.8	34.9	500.7	231.8	2.16	5.30	0.000388	47.6	0.0433	
5	Apr. 4, 1913	11.7	35.5	579.4	261.0	2.22	5.78	0.000445	43.8	0.0480	
6	Feb. 27, 1913	12.6	39.0	674.0	298.2	2.26	5.95	0.000452	43.5	0.0490	
7	Mar. 14, 1913	13.6	45.2	823.0	341.5	2.41	6.06	0.000375	50.5	0.0420	
	.....	13.0[1]									

Old Town Creek dredged channel near Tupelo, Mississippi (after clearing), 1914

8	Mar. 5, 1914	4.0	19.5	45.0	50.0	0.90	2.28	0.000358	31.5	0.0520	Description of channel practically the same as the above, except for remarks under *Condition*. *Condition*, all brush, vegetation, and obstructions to flow were cleared from the course of channel and for 500 feet above the upper end of the course, and for the same distance below the lower end.
9	Mar. 20, 1914	4.3	20.0	59.7	58.0	1.03	2.43	0.000350	35.3	0.0475	
10	Apr. 17, 1914	4.65	21.0	77.9	60.4	1.29	2.55	0.000377	41.6	0.0410	
11	Apr. 9, 1914	5.3	22.2	105.4	74.2	1.42	2.85	0.000292	49.2	0.0356	
12	Apr. 3, 1914	5.4	23.0	120.0	80.0	1.50	2.90	0.000312	49.8	0.0355	
13	Apr. 13, 1914	6.6	26.4	185.1	107.0	1.73	3.37	0.000271	57.4	0.0320	
14	Mar. 26, 1914	6.7	26.5	218.2	110.2	1.98	3.42	0.000305	61.4	0.0299	
15	Mar. 12, 1914	7.1	27.5	240.0	120.0	2.00	3.53	0.000290	62.5	0.0294	
16	Mar. 26, 1914	7.1	27.6	242.4	120.0	2.02	3.58	0.000279	64.0	0.0290	
17	Mar. 26, 1914	7.4	28.0	260.4	127.0	2.05	3.69	0.000267	65.3	0.0286	
18	Apr. 13, 1914	8.1	29.8	310.2	142.3	2.18	3.95	0.000271	66.6	0.0285	
19	May 5, 1914	11.6	37.5	710.2	265.0	2.68	5.70	0.000346	60.3	0.0340	
20	Mar. 30, 1914	12.4	39.7	815.6	295.5	2.76	5.95	0.000268	69.1	0.0297	
21	May 6, 1914	12.7	41.0	882.2	307.4	2.87	6.05	0.000346	62.8	0.0332	
	.....	13.0[1]									

[1] Bankful stage.

cleaned out earlier in the summer—gave a roughness coefficient of 0.047 for a hydraulic radius of 3.33 feet and a surface width of 50 feet. On Nov. 14, 1914, after the channel had been thoroughly cleaned, a coefficient of 0.036 was obtained for a hydraulic radius of 3.16 feet and a surface width of 50 feet.

3. The coefficient of roughness of the Miami River at Dayton,[1] as determined in the summer of 1915 when the bottom of the channel was covered with weeds, was 0.0298; whereas, after the channel was cleaned of weeds, it was 0.0255.

Irregularity of Channel.—When the bed and banks of a channel are irregular, the coefficient of roughness will be higher than for a smooth channel in the same material. This accounts for the

FIG. 64.—Dredged channel of South Forked Deer River, near Jackson, Tenn.

relatively higher values of n obtained for newly dredged channels, where the bottom and sides consist of a series of ridges and depressions left by the bucket of the dredge. The effect of irregularity in the wetted perimeter is shown in Table XIX which contains the data obtained for the dredged channel of South Forked Deer River, near Jackson, Tennessee, by the United States Bureau of Public Roads. The value of n ranged from 0.031 to 0.042 for different stages. A view of the course where the measure-

[1] *Technical Report*, Part IV, Miami Conservancy District.

TABLE XIX.—SOUTH FORKED DEER RIVER DREDGED CHANNEL AT BOLIVAR ROAD, NEAR JACKSON, TENNESSEE

No.	Date of observation	Average maximum depth, feet	Average surface width, feet	Discharge, second-feet	Average cross-section, square feet	Mean velocity, feet per second	Mean hydraulic radius, feet	Slope of water surface	Coefficient in formula $V = C\sqrt{RS}$. C	Coefficient of roughness n	Description of channel
6	Dec. 13, 1916	4.3	37.6	191.6	115.9	1.66	2.89	0.000530	42.4	0.0420	*Course*, straight; 952 feet long. *Cross-section*, very little variation in shape; some variation in size. *Side slopes*, fairly regular near top, but extremely irregular and uneven near bottom. *Bottom*, very uneven and full of holes. *Soil*, heavy clay loam. *Condition*, no vegetation or obstructions in channel. *Constructed*, March, 1916. *General remarks*, high values of n are due to very rough and irregular condition of channel as originally dredged. The upper portion of channel is fairly regular.
7	Dec. 20, 1916	4.95	38.4	282.2	143.0	1.97	3.33	0.000555	45.8	0.0402	
8	Dec. 22, 1916	5.0	38.5	290.4	147.0	1.98	3.40	0.000562	45.3	0.0401	
9	Mar. 31, 1917	5.1	38.6	375.0	150.0	2.50	3.46	0.000694	51.2	0.0362	
10	Feb. 21, 1917	5.5	39.2	433.0	166.0	2.61	3.69	0.000630	54.2	0.0348	
11	Jan. 17, 1917	5.8	39.6	477.0	178.0	2.68	3.87	0.000642	53.8	0.0349	
12	Dec. 28, 1916	9.3	51.6	1,078.0	344.0	3.13	5.69	0.000477	60.2	0.0341	
13	Jan. 31, 1917	9.8	54.0	1,296.0	366.0	3.55	5.83	0.000534	63.6	0.0322	
14	Jan. 6, 1917	10.4	57.1	1,362.0	399.4	3.41	6.10	0.000436	66.1	0.0310	
	.....	11.0[1]									

[1] Bankful stage.

ments were made is shown in Fig. 64 and the irregularity of the banks is clearly seen.

The actual wetted perimeter of a channel is always greater than that shown by the standard cross-section or that obtained by soundings in existing channels, and engineers usually increase the plotted wetted perimeter by 1 or 2 per cent to allow for irregularities in the channel as constructed.

When a channel is not uniform in cross-section, but consists of alternating stretches of larger and smaller sections, the coefficient of roughness is increased. This condition is found in all natural channels and in old artificial channels, and may increase the coefficient by 0.005 or more.

Curvature.—The retardation of flow in a channel caused by curves or bends is included in the roughness coefficient. Ordinarily, the effect of curvature is slight, although in extreme cases its amount is appreciable. It is doubtful whether curvature ever increases n more than 0.002 or 0.003. There is a tendency, however, for drift to accumulate in a crooked channel, which will increase the value of n. The effect of curvature, although usually small, should not be entirely ignored in estimating the roughness coefficient.

Stage of Stream.—The coefficient of roughness varies with the stage of the stream, or the hydraulic radius, although there is no definite relation between the two factors. Some claim that n decreases as the hydraulic radius increases, and in general this is true; but under certain conditions of channel, the reverse may be true.

R. E. Horton, in the article previously referred to in which he gives values of n for various channel characteristics, states that the higher values of n obtained by measurements at low stages are due to the facts that: (1) the full area of the section used in computing n is not effective in producing discharge because of pockets or holes in the bed of the stream; and (2) at low stages there is more or less fall that is not effective as slope and that does not contribute to producing velocity.

In the investigation made by the United States Bureau of Public Roads, and reported in *Bulletin* 832, it was found:

1. That where the bed and banks of a channel were equally smooth and regular and the bottom slope uniform, n remained the same for all stages.

2. That generally n decreased as the stage increased, because of the fact that ordinarily the bottom of a channel is rougher than the sides, and that uniform bottom slopes are difficult to obtain, causing the condition pointed out by Horton.

3. That n increased as the stage increased in those channels where the lower portion of the channel was in much better condition than the upper portion.

In all probability, the decrease in n with increase in stage which has been observed at times in natural channels is due to changes in roughness of channel or to the greater retarding action of the bed at higher velocities rather than to changes in the hydraulic radius alone.

Silting and Scouring.—The value of the roughness factor may change with changes in the regularity of the wetted perimeter and the bottom profile, caused by scouring and silting; and if the velocities are high enough to cause scouring and the material is such as to be eroded readily, these factors must be considered in choosing a value of n. Through silting, an irregular wetted perimeter may become regular; and through scouring, it may become more irregular. The amount and uniformity of scouring will depend on the material. Thus, a sandy or gravelly bed will be eroded more uniformly than one of clay. The deposition of silt eroded from the uplands will tend to even up the irregularities in a channel dredged through clay. The energy used in eroding and carrying the material in suspension or rolling it along the bed will also increase the roughness coefficient.

Floodway Coefficients.—In estimating the discharge of a stream during flood periods when the stream overflows its banks and a portion of the flow is along the flood plain, the discharge is computed separately for the area in, and over, the channel and that over the flood plain; and it becomes necessary to choose a roughness coefficient for the latter area. There is little experimental data as to the value of n for floodways, and it is customary to assume a value which is somewhat larger than that chosen for the channel proper and commensurate to it.

In 1921, the Bureau of Public Roads secured some data on the flow through cleared and uncleared portions of the diversion floodway of the Little River Drainage District near Nash, Missouri. The floodway lies between parallel levees 1,120 feet apart. The main channel in the floodway has a top width of 145 feet, a bottom width of 60 feet, and a depth of 20 feet. On a straight

FIG. 65.—Uncleared floodway of the Little River Drainage District, near Nash, Mo.

FIG. 66.—Cleared floodway of the Little River Drainage District, near Nash, Mo.

TABLE XX.—HYDRAULIC ELEMENTS AND VALUES OF n IN KUTTER'S FORMULA FOR CLEARED AND UNCLEARED FLOODWAYS

Date	Mean gage height	Average depth, feet	Average surface width, feet	Discharge, second-feet	Average cross-sectional area, square feet	Wetted perimeter, feet	Hydraulic mean depth = r, feet	Slope = s	$\sqrt{rs}$	Mean velocity = v, feet per second	$c = \frac{v}{\sqrt{rs}}$	n
For cleared course of floodway												
4/1/22	337.43	5.2	828.8	4,137	4,322.6	829.8	5.21	0.0000843	0.0210	0.957	45.6	0.047
4/2/22	337.71	5.5	829.7	4,078	4,540.8	830.6	5.47	0.0000696	0.0195	0.898	46.1	0.047
4/3/22	337.18	5.0	828.1	3,141	4,101.0	829.1	4.95	0.0000540	0.0163	0.766	47.0	0.044
4/4/22	336.55	4.4	826.6	2,243	3,579.7	827.6	4.33	0.0000506	0.0148	0.627	42.4	0.048
For uncleared course of floodway												
4/1/22	336.77	4.5	824.9	2,672	3,698.0	825.8	4.48	0.0001592	0.0267	0.723	27.1	0.077
4/2/22	337.14	4.9	826.7	2,631	4,004.2	827.2	4.84	0.0001233	0.0244	0.657	26.9	0.079
4/3/22	336.75	4.5	825.4	1,875	3,682.0	825.9	4.46	0.0000798	0.0188	0.509	27.1	0.077

section of the floodway about 2.5 miles in length, two courses of 3,000 feet each were selected for the measurements. All growth and obstructions, except stumps, were removed from one of the courses and for a distance of 500 feet beyond each end. The other course was left uncleared. Velocity measurements were made with a current meter. The hydraulic elements and values of n for the two courses are given in Table XX, and views of the cleared and uncleared floodway are shown in Figs. 65 and 66. The average value of n for the cleared portion was 0.0465, and for the uncleared portion, 0.0777.

Where the flood plain of a stream is free from ridges and depressions, and the surface is cleared of all trees and brush for cultivation, the roughness coefficient for the flood plain may not be any higher than that for the main stream. This was found to be the condition on the Miami River above Tadmor.

Effect of n upon the Discharge.—The importance of care in choosing a roughness coefficient is well illustrated in Fig. 61, in which it is seen that c—and consequently V and Q—varies as n but in the opposite direction; that is, if n is increased by a given per cent, Q is decreased by approximately the same per cent. For example, an increase in n from 0.030 to 0.033—an increase of 10 per cent—decreases c from 69 to 63, a decrease of 8.7 per cent, when the slope is 0.0004 and the hydraulic radius is 6 feet. For smaller values of the hydraulic radius, the relation is more exact.

From the foregoing discussion of the factors which affect the roughness coefficient, it is evident that the choice of n requires considerable experience and good judgment. The engineer who has not had the requisite experience must rely upon the views and descriptions of channels for which values of n have been determined experimentally by others. *Bulletin* 832 of the United States Bureau of Public Roads contains views of a large number of dredged drainage ditches together with the hydraulic data for each. Also, *Technical Report*, Part IV, of the Miami Conservancy District contains a number of views, sections, and profiles of the Miami River as well as a large amount of valuable data on the flow of water in open channels. Both of these reports should be in the library of every engineer engaged in work of this nature. The technical press has contributed, from time to time, data on individual streams.

The Manning Formula.—This formula differs from the Kutter formula principally in that the velocity varies as the two-thirds

See P-119 to

TABLE XXI.—VALUES OF nv CORRESPONDING TO DIFFERENT VALUES OF r AND s IN MANNING'S FORMULA, $v = \frac{1.486}{n} r^{2/3} s^{1/2}$

To Determine v, Divide the Tabulated Valves by n

s = slope	r = hydraulic radius in feet									
	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
0.00005	0.0023	0.0036	0.0047	0.0057	0.0066	0.0075	0.0083	0.0091	0.0098	0.0105
10	0.0032	0.0051	0.0067	0.0081	0.0094	0.0106	0.0117	0.0128	0.0139	0.0149
15	0.0039	0.0062	0.0082	0.0099	0.0115	0.0130	0.0144	0.0157	0.0170	0.0182
20	0.0045	0.0072	0.0094	0.0114	0.0132	0.0150	0.0166	0.0181	0.0196	0.0210
25	0.0051	0.0080	0.0105	0.0128	0.0148	0.0167	0.0185	0.0203	0.0219	0.0235
0.00030	0.0056	0.0088	0.0115	0.0140	0.0162	0.0183	0.0203	0.0222	0.0240	0.0257
35	0.0060	0.0095	0.0125	0.0151	0.0175	0.0198	0.0219	0.0240	0.0259	0.0278
40	0.0064	0.0102	0.0133	0.0161	0.0187	0.0211	0.0234	0.0256	0.0277	0.0297
45	0.0068	0.0108	0.0141	0.0171	0.0199	0.0224	0.0249	0.0272	0.0294	0.0316
50	0.0072	0.0114	0.0149	0.0180	0.0209	0.0236	0.0262	0.0286	0.0310	0.0332
0.00055	0.0075	0.0119	0.0156	0.0189	0.0220	0.0248	0.0275	0.0300	0.0325	0.0349
60	0.0078	0.0125	0.0163	0.0198	0.0229	0.0259	0.0287	0.0314	0.0339	0.0364
65	0.0082	0.0130	0.0170	0.0206	0.0239	0.0270	0.0299	0.0327	0.0353	0.0379
70	0.0085	0.0135	0.0176	0.0213	0.0248	0.0280	0.0310	0.0339	0.0367	0.0393
75	0.0088	0.0139	0.0182	0.0221	0.0256	0.0290	0.0321	0.0351	0.0379	0.0407
0.00080	0.0091	0.0144	0.0188	0.0228	0.0265	0.0229	0.0331	0.0362	0.0392	0.0420
85	0.0093	0.0148	0.0194	0.0235	0.0273	0.0308	0.0342	0.0374	0.0404	0.0433
90	0.0096	0.0153	0.0200	0.0242	0.0281	0.0317	0.0351	0.0384	0.0416	0.0446
95	0.0099	0.0157	0.0205	0.0249	0.0289	0.0326	0.0361	0.0395	0.0427	.0458
100	0.0101	0.0161	0.0211	0.0255	0.0296	0.0334	0.0370	0.0405	0.0439	0.0470
0.0011	0.0106	0.0169	0.0221	0.0268	0.0311	0.0351	0.0389	0.0425	0.0459	0.0493
12	0.0111	0.0176	0.0231	0.0280	0.0324	0.0366	0.0406	0.0444	0.0480	0.0515
13	0.0115	0.0183	0.0240	0.0291	0.0338	0.0381	0.0422	0.0462	0.0500	0.0536
14	0.0119	0.0190	0.0249	0.0302	0.0350	0.0395	0.0438	0.0479	0.0518	0.0556
15	0.0124	0.0197	0.0258	0.0312	0.0363	0.0410	0.0454	0.0496	0.0537	0.0576
0.0016	0.0128	0.0203	0.0266	0.0323	0.0375	0.0423	0.0469	0.0512	0.0554	0.0594
17	0.0132	0.0210	0.0275	0.0333	0.0387	0.0436	0.0483	0.0528	0.0571	0.0613
18	0.0136	0.0216	0.0283	0.0342	0.0397	0.0449	0.0497	0.0543	0.0587	0.0630
19	0.0140	0.0222	0.0290	0.0352	0.0409	0.0461	0.0511	0.0558	0.0604	0.0648
20	0.0143	0.0227	0.0298	0.0361	0.0419	0.0473	0.0524	0.0537	0.0620	0.0665
0.0025	0.0160	0.0254	0.0333	0.0403	0.0468	0.0529	0.0586	0.0641	0.0693	0.0743
30	0.0175	0.0278	0.0365	0.0442	0.0513	0.0579	0.0642	0.0702	0.0759	0.0814
35	0.0189	0.0301	0.0394	0.0477	0.0554	0.0625	0.0693	0.0758	0.0820	0.0879
40	0.0202	0.0321	0.0421	0.0510	0.0592	0.0669	0.0741	0.0810	0.0876	0.0940
45	0.0215	0.0341	0.0447	0.0541	0.0628	0.0709	0.0786	0.0859	0.0929	0.0997
0.0050	0.0226	0.0359	0.0471	0.0570	0.0662	0.0748	0.0828	0.0906	0.0980	0.1051
55	0.0237	0.0377	0.0494	0.0598	0.0694	0.0784	0.0869	0.0950	0.1027	0.1102
60	0.0248	0.0394	0.0516	0.0625	0.0725	0.0819	0.0908	0.0992	0.1073	0.1151
65	0.0258	0.0410	0.0537	0.0650	0.0755	0.0852	0.0945	0.1033	0.1117	0.1198
70	0.0268	0.0425	0.0557	0.0675	0.0783	0.0884	0.0980	0.1071	0.1159	0.1243
0.0075	0.0277	0.0440	0.0577	0.0699	0.0811	0.0916	0.1015	0.1109	0.1200	0.1287
80	0.0286	0.0455	0.0596	0.0722	0.0837	0.0946	0.1048	0.1145	0.1239	0.1329
85	0.0295	0.0469	0.0614	0.0744	0.0863	0.0975	0.1080	0.1181	0.1277	0.1370
90	0.0304	0.0482	0.0632	0.0765	0.0888	0.1003	0.1111	0.1215	0.1314	0.1410
95	0.0312	0.0495	0.0649	0.0786	0.0912	0.1030	0.1142	0.1248	0.1350	0.1448
0.0100	0.0320	0.0508	0.0666	0.0807	0.0936	0.1057	0.1172	0.1281	0.1385	0.1486

[1] Reproduced From King's "Handbook of Hydraulics"

TABLE XXI.—VALUES OF nv CORRESPONDING TO DIFFERENT VALUES OF r AND s IN MANNING'S FORMULA, $v = \frac{1.486}{n} r^{2/3} s^{1/2}$.—(*Continued*)

To Determine v, Divide the Tabulated Values by n

s = slope	r = hydraulic radius in feet									
	1.1	1.2	1.3	1.4	1.5	1.6	1.7	1.8	1.9	2.0
0.00005	0.0112	0.0119	0.0125	0.0132	0.0138	0.0144	0.0150	0.0156	0.0161	0.0167
10	0.0158	0.0168	0.0177	0.0186	0.0195	0.0203	0.0212	0.0220	0.0228	0.0236
15	0.0194	0.0206	0.0217	0.0228	0.0239	0.0249	0.0259	0.0269	0.0279	0.0289
20	0.0224	0.0237	0.0250	0.0263	0.0275	0.0288	0.0299	0.0311	0.0322	0.0334
25	0.0250	0.0265	0.0280	0.0294	0.0308	0.0321	0.0335	0.0348	0.0360	0.0373
0.00030	0.0274	0.0291	0.0307	0.0322	0.0337	0.0352	0.0367	0.0381	0.0395	0.0409
35	0.0296	0.0314	0.0331	0.0348	0.0364	0.0380	0.0396	0.0411	0.0427	0.0441
40	0.0317	0.0336	0.0354	0.0372	0.0389	0.0407	0.0423	0.0440	0.0456	0.0472
45	0.0336	0.0356	0.0376	0.0395	0.0413	0.0431	0.0449	0.0467	0.0484	0.0500
50	0.0354	0.0375	0.0396	0.0416	0.0435	0.0454	0.0473	0.0492	0.0510	0.0528
0.00055	0.0371	0.0394	0.0415	0.0436	0.0457	0.0477	0.0496	0.0516	0.0535	0.0553
60	0.0388	0.0411	0.0434	0.0456	0.0477	0.0498	0.0519	0.0539	0.0558	0.0578
65	0.0405	0.0428	0.0451	0.0474	0.0497	0.0518	0.0540	0.0561	0.0581	0.0601
70	0.0419	0.0444	0.0468	0.0492	0.0515	0.0538	0.0560	0.0582	0.0603	0.0624
75	0.0434	0.0460	0.0485	0.0509	0.0533	0.0556	0.0580	0.0602	0.0624	0.0646
0.00080	0.0448	0.0475	0.0501	0.0526	0.0551	0.0575	0.0599	0.0622	0.0645	0.0667
85	0.0462	0.0489	0.0516	0.0542	0.0568	0.0593	0.0617	0.0641	0.0665	0.0688
90	0.0475	0.0503	0.0531	0.0558	0.0584	0.0610	0.0635	0.0660	0.0684	0.0708
95	0.0488	0.0517	0.0546	0.0573	0.0600	0.0627	0.0653	0.0678	0.0703	0.0727
100	0.0501	0.0531	0.0560	0.0588	0.0616	0.0643	0.0669	0.0695	0.0721	0.0746
0.0011	0.0525	0.0557	0.0587	0.0617	0.0646	0.0674	0.0702	0.0729	0.0756	0.0782
12	0.0549	0.0581	0.0613	0.0644	0.0675	0.0704	0.0733	0.0762	0.0790	0.0817
13	0.0571	0.0605	0.0638	0.0671	0.0702	0.0733	0.0763	0.0793	0.0822	0.0851
14	0.0593	0.0628	0.0662	0.0696	0.0729	0.0761	0.0792	0.0823	0.0853	0.0883
15	0.0613	0.0650	0.0686	0.0720	0.0754	0.0787	0.0820	0.0852	0.0883	0.0914
0.0016	0.0633	0.0671	0.0708	0.0744	0.0779	0.0813	0.0847	0.0880	0.0912	0.0944
17	0.0653	0.0692	0.0730	0.0767	0.0803	0.0838	0.0873	0.0907	0.0940	0.0973
18	0.0672	0.0712	0.0751	0.0789	0.0826	0.0862	0.0898	0.0933	0.0967	0.1001
19	0.0690	0.0732	0.0772	0.0811	0.0849	0.0886	0.0923	0.0959	0.0994	0.1028
20	0.0708	0.0751	0.0792	0.0832	0.0871	0.0909	0.0947	0.0984	0.1020	0.1055
0.0025	0.0792	0.0839	0.0885	0.0930	0.0974	0.1016	0.1058	0.1099	0.1140	0.1180
30	0.0867	0.0919	0.0970	0.1019	0.1067	0.1113	0.1159	0.1204	0.1249	0.1292
35	0.0937	0.0993	0.1047	0.1100	0.1152	0.1203	0.1252	0.1301	0.1349	0.1396
40	0.1001	0.1061	0.1119	0.1176	0.1231	0.1286	0.1339	0.1391	0.1442	0.1492
45	0.1062	0.1126	0.1188	0.1248	0.1306	0.1364	0.1420	0.1475	0.1529	0.1582
0.0050	0.1120	0.1187	0.1252	0.1315	0.1377	0.1438	0.1497	0.1555	0.1612	0.1668
55	0.1174	0.1245	0.1313	0.1379	0.1444	0.1508	0.1570	0.1631	0.1691	0.1749
60	0.1227	0.1300	0.1371	0.1441	0.1509	0.1575	0.1640	0.1703	0.1766	0.1827
65	0.1277	0.1353	0.1427	0.1499	0.1570	0.1639	0.1707	0.1773	0.1838	0.1902
70	0.1325	0.1404	0.1481	0.1556	0.1629	0.1701	0.1771	0.1840	0.1908	0.1974
0.0075	0.1371	0.1453	0.1533	0.1611	0.1687	0.1761	0.1833	0.1904	0.1974	0.2043
80	0.1416	0.1501	0.1583	0.1664	0.1742	0.1818	0.1893	0.1967	0.2039	0.2110
85	0.1460	0.1547	0.1632	0.1715	0.1795	0.1874	0.1952	0.2027	0.2102	0.2175
90	0.1502	0.1592	0.1679	0.1764	0.1847	0.1929	0.2008	0.2086	0.2163	0.2238
95	0.1544	0.1636	0.1726	0.1813	0.1898	0.1981	0.2063	0.2143	0.2222	0.2299
0.0100	0.1584	0.1678	0.1770	0.1860	0.1947	0.2033	0.2117	0.2199	0.2280	0.2359

TABLE XXI.—VALUES OF nv CORRESPONDING TO DIFFERENT VALUES OF r AND s IN MANNING'S FORMULA, $v = \frac{1.486}{n} r^{2/3} s^{1/2}$.—*(Continued)*

To Determine v, Divide the Tabulated Values by n

s = slope	r = hydraulic radius in feet									
	2.1	2.2	2.3	2.4	2.5	2.6	2.7	2.8	2.9	3.0
0.00005	0.0172	0.0178	0.0183	0.0188	0.0194	0.0199	0.0204	0.0209	0.0214	0.0219
10	0.0244	0.0251	0.0259	0.0266	0.0274	0.0281	0.0288	0.0295	0.0302	0.0309
15	0.0299	0.0308	0.0317	0.0326	0.0335	0.0344	0.0353	0.0362	0.0370	0.0379
20	0.0345	0.0356	0.0366	0.0377	0.0387	0.0397	0.0407	0.0417	0.0427	0.0437
25	0.0385	0.0397	0.0409	0.0421	0.0433	0.0444	0.0456	0.0467	0.0478	0.0489
0.00030	0.0422	0.0435	0.0449	0.0462	0.0474	0.0487	0.0499	0.0511	0.0523	0.0535
35	0.0456	0.0470	0.0484	0.0498	0.0512	0.0526	0.0539	0.0552	0.0565	0.0578
40	0.0487	0.0503	0.0518	0.0533	0.0548	0.0562	0.0576	0.0590	0.0604	0.0618
45	0.0517	0.0533	0.0549	0.0565	0.0581	0.0596	0.0611	0.0626	0.0641	0.0656
50	0.0545	0.0562	0.0579	0.0596	0.0612	0.0628	0.0644	0.0660	0.0676	0.0691
0.00055	0.0572	0.0590	0.0607	0.0625	0.0642	0.0659	0.0676	0.0692	0.0709	0.0725
60	0.0596	0.0616	0.0634	0.0653	0.0671	0.0688	0.0706	0.0723	0.0740	0.0757
65	0.0621	0.0641	0.0660	0.0679	0.0698	0.0716	0.0735	0.0753	0.0771	0.0788
70	0.0645	0.0665	0.0685	0.0705	0.0724	0.0743	0.0762	0.0781	0.0800	0.0818
75	0.0667	0.0688	0.0709	0.0730	0.0750	0.0770	0.0789	0.0809	0.0828	0.0847
0.00080	0.0689	0.0711	0.0732	0.0753	0.0774	0.0795	0.0815	0.0835	0.0855	0.0874
85	0.0711	0.0733	0.0755	0.0777	0.0798	0.0819	0.0840	0.0861	0.0881	0.0901
90	0.0731	0.0754	0.0777	0.0799	0.0821	0.0843	0.0864	0.0886	0.0907	0.0927
95	0.0751	0.0775	0.0798	0.0821	0.0844	0.0866	0.0888	0.0910	0.0931	0.0953
100	0.0771	0.0795	0.0819	0.0842	0.0866	0.0889	0.0911	0.0934	0.0956	0.0978
0.0011	0.0808	0.0834	0.0859	0.0884	0.0908	0.0932	0.0956	0.0979	0.1002	0.1025
12	0.0844	0.0871	0.0897	0.0923	0.0948	0.0973	0.0998	0.1023	0.1047	0.1071
13	0.0879	0.0906	0.0934	0.0960	0.0987	0.1013	0.1039	0.1064	0.1090	0.1115
14	0.0912	0.0941	0.0969	0.0997	0.1024	0.1051	0.1078	0.1105	0.1131	0.1157
15	0.0944	0.0974	0.1003	0.1032	0.1060	0.1088	0.1116	0.1143	0.1170	0.1197
0.0016	0.0975	0.1006	0.1036	0.1066	0.1095	0.1124	0.1153	0.1181	0.1209	0.1236
17	0.1005	0.1036	0.1068	0.1098	0.1129	0.1159	0.1188	0.1218	0.1246	0.1274
18	0.1034	0.1066	0.1099	0.1130	0.1161	0.1192	0.1222	0.1252	0.1282	0.1311
19	0.1062	0.1096	0.1129	0.1161	0.1193	0.1225	0.1256	0.1287	0.1317	0.1347
20	0.1090	0.1124	0.1158	0.1191	0.1224	0.1257	0.1289	0.1320	0.1352	0.1382
0.0025	0.1218	0.1257	0.1295	0.1332	0.1369	0.1405	0.1441	0.1476	0.1511	0.1546
30	0.1335	0.1377	0.1418	0.1459	0.1499	0.1539	0.1578	0.1617	0.1655	0.1693
35	0.1442	0.1487	0.1532	0.1576	0.1619	0.1662	0.1705	0.1747	0.1788	0.1829
40	0.1541	0.1590	0.1638	0.1685	0.1731	0.1777	0.1822	0.1867	0.1911	0.1955
45	0.1635	0.1686	0.1737	0.1787	0.1836	0.1885	0.1933	0.1980	0.2027	0.2074
0.0050	0.1723	0.1777	0.1831	0.1884	0.1936	0.1987	0.2037	0.2087	0.2137	0.2186
55	0.1807	0.1864	0.1920	0.1976	0.2030	0.2084	0.2137	0.2189	0.2241	0.2292
60	0.1888	0.1947	0.2006	0.2063	0.2120	0.2177	0.2232	0.2287	0.2341	0.2394
65	0.1965	0.2027	0.2088	0.2148	0.2207	0.2265	0.2323	0.2380	0.2436	0.2492
70	0.2039	0.2103	0.2166	0.2229	0.2290	0.2351	0.2411	0.2470	0.2528	0.2586
0.0075	0.2110	0.2177	0.2242	0.2307	0.2371	0.2433	0.2495	0.2557	0.2617	0.2677
80	0.2180	0.2248	0.2316	0.2383	0.2448	0.2513	0.2577	0.2640	0.2703	0.2765
85	0.2247	0.2317	0.2397	0.2456	0.2524	0.2591	0.2657	0.2722	0.2786	0.2850
90	0.2312	0.2385	0.2456	0.2527	0.2597	0.2666	0.2734	0.2801	0.2867	0.2932
95	0.2375	0.2450	0.2524	0.2596	0.2668	0.2739	0.2808	0.2877	0.2946	0.3013
0.0100	0.2437	0.2514	0.2589	0.2664	0.2737	0.2810	0.2881	0.2952	0.3022	0.3091

TABLE XXI.—VALUES OF nv CORRESPONDING TO DIFFERENT VALUES OF r AND s IN MANNING'S FORMULA, $v = \frac{1.486}{n} r^{2/3} s^{1/2}$.—*(Continued)*

To Determine v, Divide the Tabulated Values by n

s = slope	r = hydraulic radius in feet									
	3.1	3.2	3.3	3.4	3.5	3.6	3.7	3.8	3.9	4.0
0.00005	0.0223	0.0228	0.0233	0.0238	0.0242	0.0247	0.0251	0.0256	0.0260	0.0265
10	0.0316	0.0323	0.0329	0.0336	0.0343	0.0349	0.0356	0.0362	0.0368	0.0374
15	0.0387	0.0395	0.0403	0.0412	0.0420	0.0428	0.0435	0.0443	0.0451	0.0459
20	0.0447	0.0456	0.0466	0.0475	0.0484	0.0494	0.0503	0.0512	0.0521	0.0530
25	0.0500	0.0510	0.0521	0.0531	0.0542	0.0552	0.0562	0.0572	0.0582	0.0592
0.00030	0.0547	0.0559	0.0571	0.0582	0.0593	0.0605	0.0616	0.0627	0.0638	0.0649
35	0.0591	0.0604	0.0616	0.0629	0.0641	0.0653	0.0665	0.0677	0.0689	0.0701
40	0.0632	0.0645	0.0659	0.0672	0.0685	0.0698	0.0711	0.0724	0.0736	0.0749
45	0.0670	0.0685	0.0699	0.0713	0.0727	0.0741	0.0754	0.0758	0.0781	0.0794
50	0.0706	0.0722	0.0737	0.0751	0.0766	0.0781	0.0795	0.0809	0.0823	0.0837
0.00055	0.0741	0.0757	0.0773	0.0788	0.0803	0.0819	0.0834	0.0849	0.0864	0.0878
60	0.0774	0.0791	0.0807	0.0823	0.0839	0.0855	0.0871	0.0886	0.0902	0.0917
65	0.0806	0.0823	0.0840	0.0857	0.0873	0.0890	0.0906	0.0923	0.0939	0.0955
70	0.0836	0.0854	0.0872	0.0889	0.0906	0.0924	0.0941	0.0957	0.0974	0.0991
75	0.0865	0.0884	0.0902	0.0920	0.0938	0.0956	0.0974	0.0991	0.1008	0.1026
0.00080	0.0894	0.0913	0.0932	0.0950	0.0969	0.0987	0.1005	0.1024	0.1041	0.1059
85	0.0921	0.0941	0.0960	0.0980	0.0999	0.1018	0.1036	0.1055	0.1073	0.1092
90	0.0948	0.0968	0.0988	0.1008	0.1028	0.1047	0.1066	0.1086	0.1105	0.1123
95	0.0974	0.0995	0.1015	0.1036	0.1056	0.1076	0.1096	0.1115	0.1135	0.1154
100	0.0999	0.1021	0.1042	0.1063	0.1083	0.1104	0.1124	0.1144	0.1164	0.1184
0.0011	0.1048	0.1070	0.1093	0.1114	0.1136	0.1158	0.1179	0.1200	0.1221	0.1242
12	0.1094	0.1118	0.1141	0.1164	0.1187	0.1209	0.1231	0.1254	0.1275	0.1297
13	0.1139	0.1164	0.1188	0.1212	0.1235	0.1259	0.1282	0.1305	0.1328	0.1350
14	0.1182	0.1207	0.1232	0.1257	0.1282	0.1306	0.1330	0.1354	0.1378	0.1401
15	0.1224	0.1250	0.1276	0.1301	0.1327	0.1352	0.1377	0.1401	0.1426	0.1450
0.0016	0.1264	0.1291	0.1318	0.1344	0.1370	0.1396	0.1422	0.1447	0.1473	0.1498
17	0.1303	0.1331	0.1358	0.1385	0.1412	0.1439	0.1466	0.1492	0.1518	0.1544
18	0.1340	0.1369	0.1397	0.1426	0.1453	0.1481	0.1508	0.1535	0.1562	0.1589
19	0.1377	0.1407	0.1436	0.1465	0.1493	0.1522	0.1550	0.1577	0.1605	0.1633
20	0.1413	0.1443	0.1473	0.1503	0.1532	0.1561	0.1590	0.1618	0.1647	0.1675
0.0025	0.1580	0.1614	0.1647	0.1680	0.1713	0.1745	0.1777	0.1809	0.1841	0.1872
30	0.1730	0.1768	0.1804	0.1840	0.1876	0.1912	0.1947	0.1982	0.2017	0.2051
35	0.1869	0.1909	0.1949	0.1988	0.2027	0.2065	0.2103	0.2141	0.2178	0.2215
40	0.1998	0.2041	0.2083	0.2125	0.2167	0.2208	0.2248	0.2289	0.2329	0.2368
45	0.2119	0.2165	0.2210	0.2254	0.2298	0.2342	0.2385	0.2427	0.2470	0.2512
0.0050	0.2234	0.2282	0.2329	0.2376	0.2422	0.2468	0.2514	0.2559	0.2603	0.2648
55	0.2343	0.2393	0.2443	0.2492	0.2541	0.2589	0.2636	0.2684	0.2731	0.2777
60	0.2447	0.2500	0.2552	0.2603	0.2654	0.2704	0.2754	0.2803	0.2852	0.2901
65	0.2548	0.2602	0.2656	0.2709	0.2762	0.2814	0.2866	0.2917	0.2968	0.3019
70	0.2643	0.2700	0.2756	0.2811	0.2866	0.2920	0.2974	0.3028	0.3080	0.3133
0.0075	0.2736	0.2795	0.2852	0.2910	0.2967	0.3023	0.3079	0.3134	0.3189	0.3242
80	0.2826	0.2886	0.2946	0.3005	0.3064	0.3122	0.3180	0.3237	0.3293	0.3349
85	0.2913	0.2975	0.3037	0.3098	0.3158	0.3218	0.3277	0.3336	0.3395	0.3452
90	0.2997	0.3061	0.3125	0.3188	0.3250	0.3311	0.3372	0.3433	0.3493	0.3552
95	0.3079	0.3145	0.3210	0.3275	0.3339	0.3402	0.3465	0.3527	0.3589	0.3650
0.0100	0.3159	0.3227	0.3294	0.3360	0.3426	0.3491	0.3555	0.3619	0.3682	0.3745

TABLE XXI.—VALUES OF nv CORRESPONDING TO DIFFERENT VALUES OF r AND s IN MANNING'S FORMULA, $v = \frac{1.486}{n} r^{2/3} s^{1/2}$.—*(Continued)*

To Determine v, Divide the Tabulated Values by n

s = slope	r = hydraulic radius in feet									
	4.2	4.4	4.6	4.8	5.0	5.2	5.4	5.6	5.8	6.0
0.00005	0.0274	0.0282	0.0291	0.0299	0.0307	0.0315	0.0323	0.0331	0.0339	0.0347
10	0.0387	0.0399	0.0411	0.0423	0.0435	0.0446	0.0457	0.0469	0.0480	0.0491
15	0.0474	0.0489	0.0503	0.0518	0.0532	0.0546	0.0560	0.0574	0.0588	0.0601
20	0.0547	0.0564	0.0581	0.0598	0.0615	0.0631	0.0647	0.0663	0.0678	0.0694
25	0.0612	0.0631	0.0650	0.0669	0.0687	0.0705	0.0723	0.0741	0.0759	0.0776
0.00030	0.0670	0.0691	0.0712	0.0732	0.0753	0.0773	0.0792	0.0812	0.0831	0.0850
35	0.0724	0.0747	0.0769	0.0791	0.0813	0.0834	0.0856	0.0877	0.0897	0.0918
40	0.0774	0.0798	0.0822	0.0846	0.0869	0.0892	0.0915	0.0937	0.0959	0.0981
45	0.0821	0.0846	0.0872	0.0897	0.0922	0.0946	0.0970	0.0994	0.1018	0.1041
50	0.0865	0.0892	0.0919	0.0946	0.0972	0.0997	0.1023	0.1048	0.1073	0.1097
0.00055	0.0907	0.0936	0.0964	0.0992	0.1019	0.1046	0.1073	0.1099	0.1125	0.1152
60	0.0948	0.0977	0.1007	0.1036	0.1064	0.1092	0.1120	0.1148	0.1175	0.1202
65	0.0986	0.1017	0.1048	0.1078	0.1108	0.1137	0.1166	0.1195	0.1223	0.1251
70	0.1024	0.1056	0.1087	0.1119	0.1150	0.1180	0.1210	0.1240	0.1269	0.1298
75	0.1059	0.1093	0.1125	0.1158	0.1190	0.1222	0.1253	0.1284	0.1314	0.1344
0.00080	0.1094	0.1129	0.1163	0.1196	0.1229	0.1262	0.1294	0.1325	0.1357	0.1388
85	0.1128	0.1163	0.1198	0.1233	0.1267	0.1300	0.1334	0.1366	0.1399	0.1431
90	0.1161	0.1197	0.1233	0.1269	0.1304	0.1338	0.1372	0.1406	0.1439	0.1472
95	0.1192	0.1230	0.1267	0.1303	0.1339	0.1375	0.1410	0.1444	0.1479	0.1512
100	0.1223	0.1262	0.1300	0.1337	0.1374	0.1410	0.1446	0.1482	0.1517	0.1553
0.0011	0.1283	0.1323	0.1363	0.1402	0.1441	0.1479	0.1517	0.1554	0.1591	0.1627
12	0.1340	0.1382	0.1424	0.1465	0.1505	0.1545	0.1584	0.1623	0.1662	0.1700
13	0.1395	0.1439	0.1482	0.1525	0.1567	0.1608	0.1649	0.1690	0.1730	0.1769
14	0.1447	0.1493	0.1538	0.1582	0.1626	0.1669	0.1711	0.1753	0.1795	0.1836
15	0.1498	0.1545	0.1592	0.1638	0.1683	0.1727	0.1771	0.1815	0.1858	0.1900
0.0016	0.1547	0.1596	0.1644	0.1691	0.1738	0.1784	0.1830	0.1874	0.1919	0.1963
17	0.1595	0.1645	0.1695	0.1743	0.1792	0.1839	0.1886	0.1932	0.1978	0.2023
18	0.1641	0.1693	0.1744	0.1794	0.1844	0.1892	0.1941	0.1988	0.2035	0.2082
19	0.1686	0.1739	0.1792	0.1843	0.1894	0.1944	0.1994	0.2043	0.2091	0.2139
20	0.1730	0.1784	0.1838	0.1891	0.1943	0.1995	0.2046	0.2096	0.2145	0.2194
0.0025	0.1934	0.1995	0.2055	0.2114	0.2173	0.2230	0.2287	0.2343	0.2399	0 2454
30	0.2119	0.2186	0.2251	0.2316	0.2380	0.2443	0.2505	0.2567	0.2628	0 2688
35	0.2289	0.2361	0.2432	0.2502	0.2571	0.2639	0.2706	0.2772	0.2838	0 2903
40	0.2447	0.2524	0.2600	0.2674	0.2748	0.2821	0.2893	0.2964	0.3034	0 3103
45	0.2595	0.2677	0.2757	0.2837	0.2915	0.2992	0.3068	0.3144	0.3218	0.3292
0.0050	0.2735	0.2821	0.2906	0.2990	0.3072	0.3154	0.3234	0.3314	0.3392	0.3470
55	0.2869	0.2959	0.3048	0.3136	0.3222	0.3308	0.3392	0.3475	0.3558	0 3639
60	0.2996	0.3091	0.3184	0.3275	0.3366	0.3455	0.3543	0.3630	0.3716	0.3801
65	0.3119	0.3217	0.3314	0.3409	0.3503	0.3596	0.3688	0.3778	0.3868	0 3956
70	0.3237	0.3338	0.3439	0.3538	0.3635	0.3732	0.3827	0.3921	0.4014	0.4105
0.0075	0.3350	0.3456	0.3560	0.3662	0.3763	0.3863	0.3961	0.4058		
80	0.3460	0.3569	0.3676	0.3782	0.3886	0.3989	0.4091			
85	0.3566	0.3679	0.3790	0.3898	0.4006					
90	0.3670	0.3785	0.3899	0.4012						
95	0.3770	0.3889	0.4006							
0.0100	0.3868	0.3990								

TABLE XXI.—VALUES OF nv CORRESPONDING TO DIFFERENT VALUES OF r AND s IN MANNING'S FORMULA, $v = \frac{1.486}{n} r^{2/3} s^{1/2}$.—(*Continued*)

To Determine v, Divide the Tabulated Values by n

s = slope	r = hydraulic radius in feet									
	6.2	6.4	6.6	6.8	7.0	7.2	7.4	7.6	7.8	8.0
0.00005	0.0355	0.0362	0.0370	0.0377	0.0385	0.0392	0.0399	0.0406	0.0413	0.0420
10	0.0502	0.0512	0.0523	0.0533	0.0544	0.0554	0.0564	0.0574	0.0584	0.0594
15	0.0614	0.0627	0.0640	0.0653	0.0666	0.0679	0.0691	0.0704	0.0716	0.0728
20	0.0709	0.0724	0.0739	0.0754	0.0769	0.0784	0.0798	0.0812	0.0826	0.0841
25	0.0793	0.0810	0.0827	0.0843	0.0860	0.0876	0.0892	0.0908	0.0924	0.0940
0.00030	0.0869	0.0887	0.0906	0.0924	0.0942	0.0960	0.0977	0.0995	0.1012	0.1030
35	0.0938	0.0958	0.0978	0.0998	0.1017	0.1037	0.1056	0.1075	0.1093	0.1112
40	0.1003	0.1025	0.1046	0.1067	0.1088	0.1108	0.1129	0.1149	0.1169	0.1189
45	0.1064	0.1087	0.1109	0.1132	0.1154	0.1175	0.1197	0.1219	0.1240	0.1261
50	0.1121	0.1145	0.1169	0.1193	0.1216	0.1239	0.1262	0.1284	0.1307	0.1329
0.00055	0.1176	0.1201	0.1226	0.1251	0.1275	0.1299	0.1323	0.1347	0.1371	0.1394
60	0.1229	0.1255	0.1281	0.1307	0.1332	0.1357	0.1382	0.1407	0.1432	0.1456
65	0.1279	0.1306	0.1333	0.1360	0.1386	0.1413	0.1439	0.1464	0.1490	0.1515
70	0.1327	0.1355	0.1383	0.1411	0.1439	0.1466	0.1493	0.1520	0.1546	0.1573
75	0.1374	0.1403	0.1432	0.1461	0.1489	0.1517	0.1545	0.1573	0.1601	0.1628
0.00080	0.1419	0.1449	0.1479	0.1509	0.1538	0.1567	0.1596	0.1625	0.1653	0.1681
85	0.1462	0.1493	0.1524	0.1555	0.1585	0.1615	0.1645	0.1675	0.1704	0.1733
90	0.1505	0.1537	0.1569	0.1600	0.1631	0.1662	0.1693	0.1723	0.1753	0.1783
95	0.1546	0.1579	0.1612	0.1644	0.1676	0.1708	0.1739	0.1771	0.1801	0.1832
100	0.1586	0.1620	0.1653	0.1687	0.1720	0.1752	0.1785	0.1817	0.1848	0.1880
0.0011	0.1663	0.1699	0.1734	0.1769	0.1804	0.1838	0.1872	0.1905	0.1938	0.1971
12	0.1737	0.1774	0.1811	0.1848	0.1884	0.1919	0.1955	0.1990	0.2025	0.2059
13	0.1808	0.1847	0.1885	0.1923	0.1961	0.1998	0.2035	0.2071	0.2107	0.2143
14	0.1877	0.1917	0.1956	0.1996	0.2035	0.2073	0.2111	0.2149	0.2187	0.2224
15	0.1942	0.1984	0.2025	0.2066	0.2106	0.2146	0.2186	0.2225	0.2264	0.2302
0.0016	0.2006	0.2049	0.2091	0.2134	0.2175	0.2216	0.2257	0.2298	0.2338	0.2378
17	0.2068	0.2112	0.2156	0.2199	0.2242	0.2285	0.2327	0.2368	0.2410	0.2451
18	0.2128	0.2173	0.2218	0.2263	0.2307	0.2351	0.2394	0.2437	0.2480	0.2522
19	0.2186	0.2233	0.2279	0.2325	0.2370	0.2415	0.2460	0.2504	0.2548	0.2591
20	0.2243	0.2291	0.2338	0.2385	0.2432	0.2478	0.2524	0.2569	0.2614	0.2658
0.0025	0.2508	0.2561	0.2614	0.2667	0.2719	0.2770	0.2822	0.2872	0.2922	0.2972
30	0.2747	0.2806	0.2864	0.2921	0.2978	0.3035	0.3091	0.3146	0.3201	0.3256
35	0.2967	0.3030	0.3093	0.3156	0.3217	0.3278	0.3338	0.3398	0.3458	0.3517
40	0.3172	0.3240	0.3307	0.3373	0.3439	0.3504	0.3569	0.3633	0.3696	0.3759
45	0.3364	0.3436	0.3507	0.3578	0.3648	0.3717	0.3785	0.3853	0.3921	0.3987
0.0050	0.3546	0.3622	0.3697	0.3771	0.3845	0.3918	0.3990	0.4062	0.4133	0.4203
55	0.3719	0.3799	0.3878	0.3956	0.4033	0.4109	0.4185			
60	0.3885	0.3968	0.4050	0.4132						
65	0.4043	0.4130								
70										

TABLE XXI.—VALUES OF nv CORRESPONDING TO DIFFERENT VALUES OF r AND s IN MANNING'S FORMULA, $v = \frac{1.486}{n} r^{2/3} s^{1/2}$.—(*Continued*)

To Determine v, Divide the Tabulated values by n

s = slope	r = hydraulic radius in feet									
	8.2	8.4	8.6	8.8	9.0	9.2	9.4	9.6	9.8	10.0
0.00005	0.0427	0.0434	0.0441	0.0448	0.0445	0.0461	0.0468	0.0475	0.0481	0.0488
10	0.0604	0.0614	0.0624	0.0633	0.0643	0.0653	0.0662	0.0671	0.0681	0.0690
15	0.0740	0.0752	0.0764	0.0776	0.0787	0.0799	0.0811	0.0822	0.0833	0.0845
20	0.0855	0.0868	0.0882	0.0896	0.0909	0.0923	0.0936	0.0949	0.0962	0.0975
25	0.0955	0.0971	0.0986	0.1002	0.1017	0.1032	0.1047	0.1061	0.1076	0.1091
0.00030	0.1047	0.1064	0.1080	0.1097	0.1114	0.1130	0.1146	0.1163	0.1179	0.1195
35	0.1131	0.1149	0.1167	0.1185	0.1203	0.1221	0.1238	0.1256	0.1273	0.1290
40	0.1209	0.1228	0.1248	0.1267	0.1286	0.1305	0.1324	0.1343	0.1361	0.1380
45	0.1282	0.1303	0.1323	0.1344	0.1364	0.1384	0.1404	0.1424	0.1444	0.1463
50	0.1351	0.1373	0.1395	0.1416	0.1438	0.1459	0.1480	0.1501	0.1522	0.1542
0.00055	0.1417	0.1440	0.1463	0.1485	0.1508	0.1530	0.1552	0.1574	0.1596	0.1618
60	0.1480	0.1504	0.1528	0.1552	0.1575	0.1598	0.1621	0.1644	0.1667	0.1690
65	0.1541	0.1566	0.1590	0.1615	0.1639	0.1664	0.1687	0.1711	0.1735	0.1759
70	0.1599	0.1625	0.1650	0.1676	0.1701	0.1726	0.1751	0.1776	0.1801	0.1825
75	0.1655	0.1682	0.1708	0.1735	0.1761	0.1787	0.1813	0.1838	0.1864	0.1889
0.00080	0.1709	0.1737	0.1764	0.1792	0.1819	0.1845	0.1872	0.1899	0.1925	0.1951
85	0.1762	0.1790	0.1819	0.1847	0.1875	0.1902	0.1930	0.1957	0.1984	0.2011
90	0.1813	0.1842	0.1871	0.1900	0.1929	0.1957	0.1986	0.2014	0.2042	0.2069
95	0.1863	0.1893	0.1923	0.1952	0.1982	0.2011	0.2040	0.2069	0.2098	0.2126
100	0.1911	0.1942	0.1973	0.2003	0.2033	0.2063	0.2093	0.2123	0.2152	0.2181
0.0011	0.2004	0.2037	0.2069	0.2101	0.2133	0.2164	0.2195	0.2226	0.2257	0.2288
12	0.2093	0.2127	0.2161	0.2194	0.2227	0.2260	0.2293	0.2325	0.2357	0.2389
13	0.2179	0.2214	0.2249	0.2284	0.2318	0.2352	0.2386	0.2420	0.2454	0.2487
14	0.2261	0.2298	0.2334	0.2370	0.2406	0.2441	0.2477	0.2512	0.2546	0.2581
15	0.2340	0.2378	0.2416	0.2453	0.2490	0.2527	0.2563	0.2600	0.2636	0.2671
0.0016	0.2417	0.2456	0.2495	0.2534	0.2572	0.2610	0.2648	0.2685	0.2722	0.2759
17	0.2491	0.2532	0.2572	0.2612	0.2651	0.2690	0.2729	0.2768	0.2806	0.2844
18	0.2564	0.2605	0.2646	0.2687	0.2728	0.2768	0.2808	0.2848	0.2887	0.2926
19	0.2634	0.2677	0.2719	0.2761	0.2803	0.2844	0.2885	0.2926	0.2966	0.3007
20	0.2702	0.2746	0.2790	0.2833	0.2875	0.2918	0.2960	0.3002	0.3044	0.3085
0.0025	0.3021	0.3070	0.3119	0.3167	0.3215	0.3262	0.3309	0.3356	0.3403	0.3449
30	0.3310	0.3363	0.3417	0.3469	0.3522	0.3574	0.3625	0.3676	0.3727	0.3778
35	0.3575	0.3633	0.3690	0.3747	0.3804	0.3860	0.3916	0.3971	0.4026	0.4081
40	0.3822	0.3884	0.3945	0.4006	0.4066	0.4127	0.4186	0.4245		
45	0.4054	0.4119	0.4184							

power instead of the one-half power of the hydraulic radius. As now used, the Manning formula is:

$$V = \frac{1.486}{n} r^{2/3} s^{1/2} \tag{16}$$

in which n has the same value as in Kutter's formula. When r is 3.28 feet, the Kutter and the Manning formulas give the same results. When r is less than 3.28 feet, the Manning formula gives slightly larger values than the Kutter formula for the values of n and s ordinarily used in the design of earth channels. When r is greater than 3.28 feet, the reverse is generally true, as is shown in Figs. 67 and 68. The Manning formula is readily

solved by the use of Table XXI, which gives values of nV for different values of r and s, or by the use of the diagram in Fig. 75.

Bazin's Formula.—The first Bazin formula was proposed in 1865, 4 years before that of Ganguillet and Kutter. This formula was the result of an extensive series of experiments made from 1855 to 1862 by Darcy and Bazin, two distinguished French hydraulicians. Thirty-two years later, Bazin published his second formula, which is the one now used. Expressed in English units, the formula is:

$$V = \frac{87}{0.552 + \frac{m}{\sqrt{r}}}\sqrt{rs} \qquad (17)$$

in which m is the roughness coefficient, and the other symbols have the same significance as in the preceding formulas. Values of m for various surfaces, as proposed by Bazin, are as follows:

1. Very smooth surfaces, cement, planed wood, etc	0.06
2. Smooth surfaces, timber, brick, cut stone, etc	0.16
3. Surfaces of rubble masonry	0.46
4. Surfaces of mixed nature, very regular sections in earth, or paved with rough stones, etc	0.85
5. Earth channels in ordinary condition	1.30
6. Earth channels offering unusual resistance	1.75

Values of c for various values of m and r are given in Table XXII.

The only essential difference between Kutter's formula and that of Bazin is that the former contains a slope correction in the coefficient, while the latter does not. Although Bazin recognized the effect of slope on the coefficient c in some of his experiments, he did not find any definite relation between them, and concluded that the inclusion of such a slope correction would complicate the formula without adding to its value.

Both the Bazin formula and the Kutter formula are of the same general form and for certain values of m, n, and s will give the same coefficient c for all values of r. The minimum value of n for which the two formulas will give the same result is 0.0156, since for smaller values a negative value of s is required in the Kutter formula. Table XXIII gives in column (1) the values of m which are given in Table XXII, and in columns (2) and (3) the corresponding values of n and s which, when substituted in the Kutter formula, give the same results as the Bazin formula.

TABLE XXII.—VALUES OF c FROM BAZIN'S FORMULA FOR USE IN THE CHEZY FORMULA, $V = c\sqrt{rs}$

r	Coefficient of roughness, m												
	0.30	0.45	0.65	0.85	1.05	1.25	1.50	1.75	2.00	2.50	3.00	3.50	4.00
1	102.1	86.8	72.4	62.1	54.3	48.3	42.4	37.8	34.1	28.5	24.5	21.5	19.1
1.5	109.2	94.6	80.4	69.8	61.7	55.3	48.9	43.9	39.8	33.6	29.0	25.5	22.8
2	113.9	100.0	86.0	75.5	67.2	60.6	53.9	48.6	44.3	37.5	32.5	28.7	25.8
2.5	117.3	104.0	90.3	79.8	71.5	64.8	58.0	52.4	47.9	40.8	35.5	31.5	28.2
3	120.0	107.2	93.9	83.4	75.1	68.3	61.4	55.7	51.0	43.6	38.1	33.8	30.4
3.5	122.1	109.8	96.8	86.5	78.2	71.3	64.3	58.5	53.7	46.1	40.4	35.9	32.3
4	123.9	112.0	99.2	89.0	80.8	73.9	66.8	61.0	56.1	48.3	42.4	37.8	34.1
5	126.8	115.5	103.2	93.3	85.2	78.3	71.1	65.2	60.1	52.1	45.9	41.1	37.2
6	127.2	118.3	106.4	96.8	88.7	81.9	74.7	68.7	63.6	55.3	49.0	43.9	39.8
8	132.2	122.3	111.3	102.0	94.2	87.5	80.4	74.3	69.1	60.6	53.9	48.6	44.2
10	134.5	125.3	114.8	106.0	98.4	91.8	84.8	78.7	73.4	64.8	58.0	52.4	47.9
12	136.2	127.6	117.6	109.1	101.7	95.3	88.3	82.3	77.0	68.3	61.3	55.7	51.0
16	138.8	130.9	121.8	113.8	106.8	100.6	93.9	87.9	82.7	73.9	66.8	61.0	56.1
20	140.5	133.3	124.8	117.2	110.6	104.6	98.0	92.2	87.1	78.3	71.1	65.2	60.1
25	142.2	135.5	127.6	120.5	114.2	108.5	102.1	96.5	91.4	82.7	75.5	69.5	64.3
30	143.4	137.2	129.7	123.0	117.0	111.5	105.3	99.8	94.9	86.3	79.1	73.0	67.8
50	146.4	141.3	135.1	129.4	124.2	119.4	113.9	108.8	104.2	96.1	89.1	83.1	77.8

It will be observed that there is a definite relation between m and n for the given values of s which is expressed by the equation

$$m = 87n - 1.$$

TABLE XXIII.[1]—RELATION BETWEEN m, n, AND s IN THE BAZIN AND THE KUTTER FORMULAS

m	n	s
(1)	(2)	(3)
0.36	0.0156	
0.46	0.0168	0.0003430
0.65	0.0190	0.0001365
0.85	0.0213	0.0000907
1.07	0.0238	0.0000706
1.30	0.0264	0.0000593
1.52	0.0290	0.0000525
1.75	0.0316	0.0000479
2.00	0.0345	0.0000443
2.40	0.0391	0.0000403
3.00	0.0460	0.0000367

[1] Reproduced from *Technical Report*, Part IV, Miami Conservancy District.

The values of m in Bazin's formula for canals and natural streams, given in King's "Handbook of Hydraulics," are shown in Table XXIV.

TABLE XXIV.—VALUES OF m FOR BAZIN'S FORMULA

Surface	Best	Good	Fair	Bad
Earth canals in good condition	0.50	0.69	0.88	1.05
Earth canals with weeds, rocks, etc	1.05	1.38	1.74	2.10
Canals excavated in rock	1.38	1.74	2.04	2.32
Natural streams in good condition	1.05	1.38	1.74	2.10
Natural streams with weeds, rocks, etc	1.74	2.43	3.48	4.86

Biel's Formula.—In 1907, Biel[1] proposed a formula for the flow of liquids and gases. It is,

$$V^2 = \frac{1{,}811rs}{0.0663 + \frac{f}{\sqrt{r}} + \frac{8.2t}{(100f + 2)V\sqrt{r}}} \qquad (18)$$

in which f is a roughness factor and t is a viscosity coefficient. The values of f for different surfaces and the coresponding values of Kutter's n, as given by Biel, are as given in Table XXV.

TABLE XXV.—VALUES OF f FOR BIEL'S FORMULA

Surface	Biel's f	Kutter's n
Smooth boards and wrought-iron pipes	0.018	0.010
New cast-iron pipes and smooth cement pipes	0.036	
Rough boards and smooth brickwork	0.054	0.012
Smooth masonry channels	0.072	0.013
Rough masonry channels	0.290	0.020
Canals in earth and regular streams	0.500	0.025
Canals and rivers with stones and weeds	0.750	0.030
Canals and rivers in bad condition	1.060	0.035

The viscosity coefficient t varies from 0.0179 for a temperature of 32°F. to 0.0097 for a temperature of 70°F. This formula is the only one which contains a temperature correction. While

[1] "Mittheilungen über Forscharbeiten," *Zeitschrift Verein deutscher Ingenieure*, Heft 44.

temperature is a factor of considerable importance in the flow of gases, its effect on the flow of water is negligible. Since the temperature term affects the value of the velocity less than 1 per cent, it is usually omitted in computing the flow in open channels.

The Barnes Formula.—A. A. Barnes, an English engineer, published a series of formulas in 1916, each of the same general form—$V = cr^x s^y$—but with different values of c, x, and y for the different classes of roughness. His formula for earth canals in average working condition and for rivers free from vegetation is,

$$V = 58.4\ r^{0.694} s^{0.496} \tag{19}$$

Since this formula does not contain a variable term to allow for changes in roughness, it cannot give satisfactory results for all canals and rivers embraced under the descriptions given above.

The Williams Formula.—Gardner S. Williams in Section 9 of Merriman's "American Civil Engineers' Pocket Book," gives the following formula for flow in open channels.

$$V = C_1 r^{0.67} s^{0.54} \tag{20}$$

This formula differs slightly from the Williams and Hazen formula used in the design of sewers.

The values of C_1 proposed for channels of varying roughness are:

$C_1 = 205$ to 185 for very smooth channels.
$C_1 = 165$ to 155 for ordinary unplaned plank.
$C_1 = 155$ to 125 for ordinary sewer crock.
$C_1 = 155$ to 120 for ordinary brick sewers.
$C_1 = 105$ to 75 for ordinary earth channels.
$C_1 = 75$ to 45 for rough natural channels.

Elliott's Formula.—In his book on "Engineering for Land Drainage," C. G. Elliott gives a simple formula, which is a modification of Eytelwein's formula. It is:

$$V = \sqrt{1.5rh} \tag{21}$$

in which h is the fall of the surface in feet per mile. This formula was proposed for use in designing drainage ditches, and gives about the same results as Kutter's formula with n equal to 0.0225.

Comparison of Velocity Formulas.—Considered as to use, the formulas which have been generally accepted by engineers are those of Kutter, Bazin, and Manning. The Kutter formula has

been used almost exclusively in the United States, although the Manning formula is gaining in popularity. The Elliott formula was much used formerly in the middle western states in the design of drainage ditches, but has been largely superceded by that of Kutter. The Bazin formula is generally used in France and to some extent in the United States. In Germany and Austria, the formulas of Siedik, Gröger, Hessle, Christen, and others are used. These formulas have not been given here because they are of doubtful merit and are not used in this country. The Manning formula is favored by the English and is extensively used in India and Australia.

Considered as to convenience, there is little choice since specially prepared tables and diagrams are available which make the solutions of the several formulas equally simple.

Considered as to accuracy, each of the formulas will give satisfactory results under ordinary conditions if the proper roughness coefficients are used. (For unusual or extreme conditions, probably some one formula has the advantage over the others. In order that a visual comparison may be had, Figs. 67 and 68 have been prepared for slopes of 0.0001 and 0.0005, respectively, and for roughness conditions corresponding to $n = 0.030$ in the Kutter formula.) For both the slopes given, Barnes' and Elliott's formulas give much larger values of the velocity than the other formulas, because both authors assume a constant roughness condition for all dredged channels, and, therefore, their formulas are not as flexible as the others. Both authors have underestimated the effect of roughness of channel upon the velocity. The satisfactory use of the other formulas depends upon a knowledge of the proper coefficients. There is more information on Kutter's n than there is on Bazin's m or Biel's f. Practically all of the recent experimental investigations on flow in open channels have been discussed in terms of Kutter's n, and this information gives Kutter's formula an undoubted advantage over those of Bazin and Biel. (Since Manning's formula uses Kutter's n and is expressed in simpler terms, many engineers advocate the use of that formula.)

The most extensive study and comparison of velocity formulas in recent years is that made by the engineers of the Miami Conservancy District in connection with the Miami flood-control project, and published in *Technical Report*, Part IV. The conclusions which were reached are as follows:

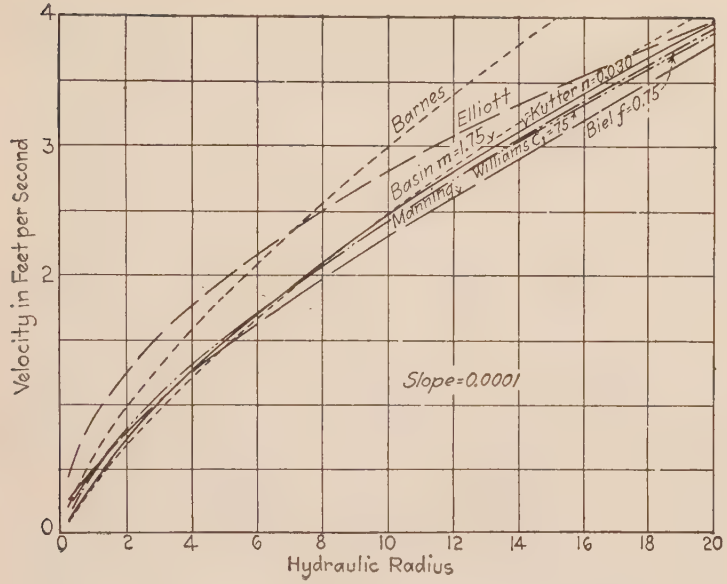


FIG. 67.—Comparison of formulas for flow of water in open channels, for slope of 0.0001.

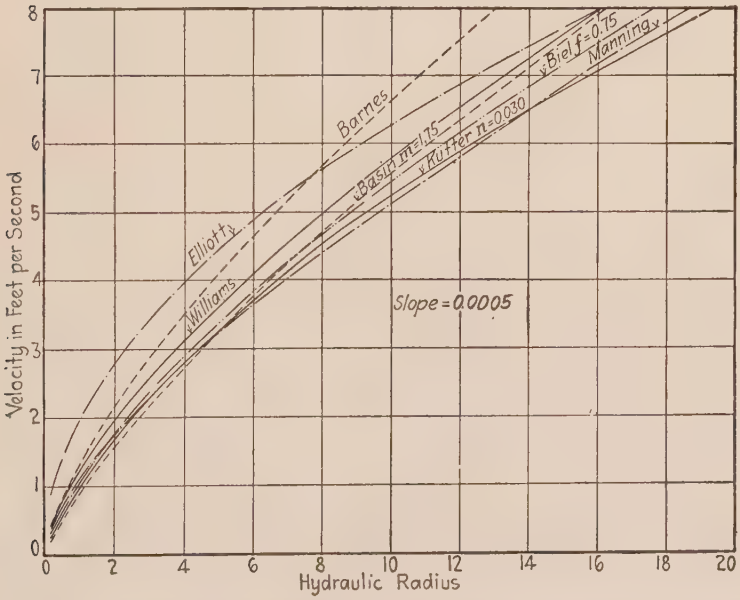


FIG. 68.—Comparison of formulas for flow of water in open channels, for slope of 0.0005.

1. Of the German formulas which have been developed on the assumption that a roughness coefficient is not necessary, not one possesses sufficient merit to warrant its adoption as a general formula.

2. It is not possible to develop a satisfactory formula for velocities in open channels without introducing therein a variable term to allow for changes in roughness.

3. No exponential formula so far advanced could be recommended for general use.

4. The effect of temperature should not be introduced into a formula for the flow of water in open channels unless its magnitude is greater than that assumed by Biel.

5. Manning's formula in its original form is practically as good as Kutter's for channels of small or ordinary dimensions, but is inferior to Kutter's for large rivers. Although its algebraic form is somewhat more simple than Kutter's equation, it does not seem advisable to adopt it for use even in ordinary instances since the latter formula is now in quite general use and, moreover, is applicable to extreme cases.

6. No definite effect of the slope on the Chezy coefficient is shown by the experimental data for small open channels.

7. Data available at present show a decrease in c with an increase in s in large rivers with flat slopes.

8. The Bazin formula is inferior to Kutter's for all types of open channels. The constancy of the factor m is less than that of the factor n in all instances.

9. Although the Kutter formula is not ideal, it is the best equation available at the present time.

Retarding Influences.—The friction between the bed and banks of a stream and the flowing water is of itself responsible for only a small part of the retardation of flow which occurs. The major portion is due to the eddies, whirls, or vortices created in the water itself by the roughness of the earth surface. These eddies consume the greatest part of the energy which otherwise would be used in producing velocity. The rougher the surface and the greater the velocity, the more vigorous are the eddies and the greater the amount of energy absorbed by them. Not only the roughness of the earth surface but also changes in the width and depth of channel, curves or bends, and obstructions of various kinds produce internal currents or vortices which cause a complicated movement of the water particles as they are carried downstream by virtue of gravity. A slight irregularity in the bed of a stream is sufficient to produce an upward current. If the bed is covered with boulders and the stream is shallow,

these upward currents will extend to the surface. At high water, the agitation at the bottom will be more violent, but may not be noticeable at the surface because of the depth. Sand waves on the bottom, however, while causing less vigorous eddy action at low water will produce upward currents during flood stages which appear at the surface as "boils." On a rising stream, the elevation of the water surface at the thread of maximum velocity is probably somewhat higher than at the sides of the stream, causing a slight surface motion toward the banks. On a falling stream, the reverse is true. Centrifugal force at curves or bends causes a movement of the surface water toward the concave bank—resulting in an inclined water surface, sloping downward from the concave bank—and a corresponding downward current along the concave bank and diagonally across the bottom to the convex bank. This helicoidal motion begins above the curve and extends below it.

The mean velocity of a stream is retarded somewhat by the presence of mud and silt in suspension, since a portion of the kinetic energy is consumed in keeping the material in suspension. Further, the viscosity and weight of the composite liquid is increased, and the effect of friction becomes greater.

The Transporting Power of Flowing Water.—Solids are transported by flowing water in three ways: (1) by continuous suspension; (2) by intermittent suspension; and (3) by rolling or skipping along the bed of the stream. In addition to the material carried in suspension, a large amount is carried in solution, but this latter material will not be considered here.

If a solid whose density is greater than unity is introduced at the surface of a stream, it tends to settle gradually to the bottom under the action of gravity as it is carried downstream. This downward force is opposed by a greater or lesser upward force created by the irregularities on the bottom, and a certain amount of material is kept in suspension. The upward pressure of the deflected currents of water varies as the square of the velocity, and, since the amount of material held in suspension is proportional to this upward pressure, it also varies as the square of the velocity; and the total quantity transported in suspension varies as the cube of the velocity. Some authorities[1] maintain that the main factor in the transportation of suspended matter is the

[1] MERRIMAN, "American Civil Engineers' Pocket Book," sec. 16, by FREDERIC R. HARRIS, Corps C. E., U. S. N.

increase in pressure near the bed of a stream caused by the decrease in velocity at that place, that is, velocity head is changed to pressure head which produces an upward motion. According to this theory, the pressure in the semi-fluid layers of sediment on the bottom is relatively high (because of the low velocity of the water) as compared with that in the layers above, and this excess pressure lifts the particles on the bottom up into the current until a pressure equilibrium is established.)

Another theory is that, since suspended matter moves at a greater velocity than the water surrounding it, the particles move in the direction of least resistance, which is toward the faster-moving water in the layers above.

In suspension, the chief factor is the upward component of motion in the complex currents, while in rolling or skipping the chief factor is the motion parallel to the bed of the stream and close to it.

The finer particles, such as clay, are carried almost entirely in continuous suspension, since even the slight upward currents caused by the low velocities at low water are sufficient to sustain them. As the depth and velocity increase, the transporting power increases rapidly and the lighter particles of sand and silt are eroded from the bottom and carried in suspension until a decrease in velocity causes them to be deposited on the bottom to await another increase in velocity. In this way they are transported by intermittent suspension and rolling. The coarser material, such as sand and gravel, may be carried in suspension at the higher velocities, or by rolling at the lower velocities if the bottom is even, or by "saltation" or skipping from point to point when the bottom is uneven.

At flood stages, small gravel is carried in suspension. The larger particles are rolled along the bottom whenever the velocity is sufficiently high. The diameters of similarly shaped solids which the current is just able to move vary as the square of the velocity and their weights theoretically as the sixth power of the velocity and actually as about the fifth power. At flood stages, large boulders are rolled along the bottom. Townsend[1] cites an instance where rocks weighing as much as 10 tons were carried downstream during a flood in a mountain stream. The material carried in suspension tends to move faster than the mean velocity of the displaced water, in the same way that a boat will

[1] TOWNSEND, COL. C. M., "River and Harbor Construction," p. 6.

move faster than the water in which it floats.[1] On the other hand, the material rolled has a much lower velocity than the water.

Just what proportion of the transported material is carried in suspension is a much debated question. In general, it can be said that for most sedimentary streams by far the larger portion is carried in suspension. There are, of course, some exceptions to this statement.

In general, the greater the velocity the greater the weight of the sediment in a unit volume of water; however, a stream will contain different amounts of sediment at the same stage and the same velocity, since the amount of sediment carried in suspension depends primarily upon the amount which is brought to the stream by its tributaries, which in turn depends upon the intensity of rainfall, the nature of the soil of the watershed, and the cultural and temperature conditions. For this reason, more material is carried in suspension on a rising stream than on a falling one.

The amount of material which a stream of given depth and velocity can transport is limited, and when the water contains its maximum amount it is said to be "saturated" or "fully charged." A greater weight of small particles can be carried than of larger particles of the same density. It is frequently stated that a stream which is not fully charged at a given velocity will pick up or erode additional material from the bed or banks until it is fully charged. This is true only under certain conditions which are discussed later. There is no definite relation between the velocity of the water and the amount of suspended matter actually present. When material has been deposited, a greater velocity is required to erode it, that is, a greater force is required to erode material than to carry it in suspension.

Since the upward currents are strongest at the bed of a stream, the amount of suspended material is greater per unit volume of water at the bottom than at the surface, especially so when the charge is sand. Horizontally, the distribution is fairly uniform from bank to bank.

Scouring and Silting of Channels.—There is unquestionably a relation between scouring and silting and velocity and depth, since for the same velocity erosion may occur in shallow water

[1] Hooker, Elon Huntington, "The Suspension of Solids in Flowing Water," *Trans. Amer. Soc. Civ. Eng.*, vol. 36, p. 322.

and silting in deep water. Kennedy[1] has given the following relation between velocity and depth for a condition of no silting nor scouring:

$$V_o = cd^m = 0.84d^{0.64} \tag{21}$$

In this equation, V_o is the "critical velocity," that is, the velocity at which silting is just prevented, and d is the depth.

This formula was derived from measurements extending over about 90 miles of the lower reaches of the Bari Doab Canal-system in India. The width of the several channels varied from 8 to 91 feet, and the depth from 2.3 to 7.3 feet. The conditions existing in the canal system were ideal for the purpose of the investigation. A practically constant discharge through a long period of years had brought the channels to a state of permanent régime, with the bed horizontal and composed of coarse sand, and with the sides nearly vertical and composed of fine sediment. The amount of silt carried in the lower reaches of the canal system was practically constant the year round. The discharges were measured and the mean velocities computed. The mean velocities thus obtained were plotted as abscissas and the corresponding depths as ordinates. The curve drawn through the points is expressed by the above formula. The results proved conclusively that for a given depth the width of channel does not enter the equation, so long as the velocity remains unchanged.

The coefficient c will vary with the nature of the suspended material, and several values are given as follows:

$c = 0.82$, for fine, light, sandy silt of northern India.
$c = 0.90$, for somewhat coarser light, sandy silt.
$c = 0.99$, for sandy loam
$c = 1.07$, for rather coarse silt, as débris of hard soils.

The curves in Fig. 69 show the effect of slope and width of channel upon scouring and silting. The critical velocity curve is plotted from Kennedy's formula and shows the velocities required at varying depths to prevent silting. The other four curves show the velocities which will exist at varying depths in a channel of trapezoidal cross-section and side slopes of one to one, computed by Kutter's formula, using $n = 0.030$. Curves A and B show that for a slope of 0.0003 and for bottom widths of 20 and 40 feet, respectively, silting will occur; also that the wider the

[1] KENNEDY, ROBT. G., "The Prevention of Silting in Irrigation Canals," *Minutes and Proc. Inst. Civ. Eng.*, vol. 119.

channel the less the tendency to silt, because of the increase in the hydraulic radius and velocity with increased width. A bottom width of 100 feet for the same slope gives a curve (not shown) almost coinciding with the critical velocity curve. Bottom widths greater than about 100 feet will cause scouring to take place. Thus, it is seen that for slopes less than about 1.5

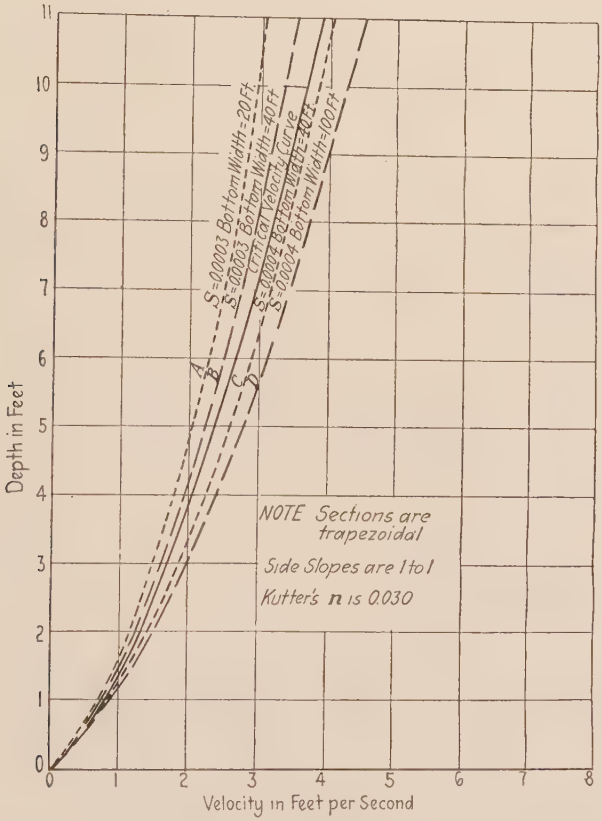


FIG. 69.—Curves showing the effect of slope and width of channel upon scouring and silting.

feet per mile and for sections commonly used in designing, some silting cannot be prevented. Curves C and D show that for a slope of 0.0004 and for bottom widths of 40 and 100 feet, respectively, scouring will occur.

In designing drainage channels where ample fall is available, the dimensions of the cross-section and the slope can be so

chosen that the velocity for various depths will agree closely with the critical velocity. (That section which will make $bd^{1.64}$ a minimum will have the least tendency to scour or silt.) Although it is impossible to prevent silting or scouring in a channel where the discharge and the amount of silt are variable, yet the relation of depth and velocity can be so chosen as to cause either silting or scouring action to predominate.

Kennedy's observations were made on irrigation canals where the bed was horizontal and the sides nearly vertical. In natural channels, and in drainage ditches a few years after construction, the beds are not horizontal, but slope toward the thread of maximum velocity, and hence the ratio of velocity to depth varies in different parts of a cross-section. This condition produces a tendency to silt where the ratio is least and to scour where it is greatest. Kennedy's formula, however, should apply approximately to these channels when the mean depth and mean velocity are used.

If a channel is not of uniform section throughout, the water will flow through the several portions with different velocities. A contraction in the channel will increase the velocity, whereas an enlargement will decrease it. The increased velocity throughout a contraction tends to enlarge the channel through erosion, while the opposite action takes place in the wider channel. Thus, other factors remaining constant, there is a tendency for an irregular channel to become uniform.

Scouring Velocities.—Numerous experiments have been made in the laboratory, and observations made on natural streams, to determine the velocities at which flowing water will just move materials of various kinds; but since such movement depends upon such factors as uniformity of materials, depth and width of channel, and volume of discharge, as well as upon velocity, the results vary for the same fineness of materials and are of uncertain value. The values of mean velocity given below are compiled from the most reliable sources of information.[1]

Observations[2] on the irrigation canals leading from the River Nile, which are excavated through alluvial earth, show that at a velocity of 2 feet per second, or less, silting occurred, while at a velocity of 2.3 feet per second there was no deposit. Velocities

[1] MERRIMAN, "American Civil Engineers' Pocket Book," 3rd ed., p. 859. KENT, "Mechanical Engineers' Pocket Book," 9th ed., p. 755.

[2] THOMAS and WATT, "The Improvement of Rivers," p. 4.

between 4 and 5 feet per second caused considerable erosion. A velocity of 3.25 feet per second was finally adopted wherever practicable. On certain irrigation canals in India, a velocity between 1.5 and 2 feet per second is found to be satisfactory.

	Feet per Second
Soft earth	0.25
Soft clay	0.50
Ordinary clay	3.00
Compact clay	5.00–6.00
Sand	1.00–2.50
Gravel	2.00–4.00
Sea pebbles (1.06 inches diameter)	2.20
Stones (size of egg)	3.30–4.00
Brickbats (4.76 cubic inches)	2.25–2.50
Slate (9.06 cubic inches)	2.75–3.00
Conglomerates	5.00
Sedimentary rocks	6.00
Solid rock	10.00

The Effect of Silting and Scouring upon a Channel.—All streams are constantly at work to establish a state of permanent régime, that is, to mold their channels by scouring and silting so that no further change will take place. This can happen only when the cross-section and slopes have become so adjusted to the velocity that the eroding force due to the velocity is equal to the adhesive force of the particles forming the bed and banks, and when all the material brought in by the tributaries can be transported by the main stream. Obviously, this ideal condition can never be realized, and even in the oldest streams constant changes of greater or less extent are taking place. In irrigation canals, such as the Bari Doab in India, where the discharge and the amount of silt brought in are constant, such a condition is possible; but in natural channels the varying conditions will not permit a permanent régime.

In a straight channel of uniform section and slope, whose design is such that the deposition of the material brought in by its tributaries and washed in by the rains is just prevented by the velocity, a small amount of erosion will occur along the bed and banks at certain stages. If the material is homogeneous and of the same tenacity, both banks will erode uniformly and the stream will remain straight. Such a condition of homogeneity, however, is not found in nature, and the banks will erode unevenly. The undermining of one bank at a soft place causes a

slide which deflects the current toward the opposite side. This curent will cut into the opposite bank a short distance downstream and cause it to cave. The material is carried a short distance and deposited, forming a bar which deflects the current toward the first bank. In this way, sinuosities in a channel are started and the force of inertia causes them to increase rapidly in extent.

In a straight channel the thread of maximum velocity is near the center of the channel; but in a curved one, centrifugal force deflects the current toward the concave bank and causes the main volume of flow to follow that bank until a straight section is reached, when the thread gradually crosses the stream and reaches the other bank at the next bend. The water surface at the concave bank is higher than at the convex bank, and the increased head causes a current to move downward along the concave bank and diagonally across the bottom to the surface at the convex bank. Some of the material eroded by the swift turbulent current along the concave bank is deposited in the slower current along the convex bank and builds up that bank so that the width of the stream remains fairly constant. A cross-section of a bend shows a steep concave bank, a deep channel near that bank, and a gently sloping bed to the convex bank. The deepest channel is found just below the point of sharpest curvature.

As a stream leaves a bend, the water gradually loses its transverse motion, and instead of depositing material on the convex bank it builds up a bar extending from the convex bank diagonally across the stream to the opposite bank. The bar or dam thus formed changes the slopes and thereby decreases the velocity above, and increases it below. On a rising stream, more material is brought to the bar than the water can transport over it, with the result that some of the material is deposited, thus raising the height of the bar and decreasing the slope, the velocity, and the scouring action above until an equilibrium is reached between the material brought to the bar and that which the water is capable of transporting. As the river falls, the increased slope below the bar increases the velocity, and scouring takes place gradually until the level of the bar is about the same as before the high water. A low-water channel is scoured through the softer deposits by the falling stream. In a wide river, this channel may change its position from flood to flood. If the river

falls gradually, the amount of scouring across the bars is greater than where the fall is sudden.

In time of flood, the flow of water through the curved portion of a stream is very turbulent, while across the bars or shoals it is much more tranquil. During periods of low water, on the other hand, the flow through the pools is slow and steady, while that over the shoals is more or less disturbed.

Thus, the bed of an alluvial stream consists of alternate depressions and ridges, which at low water produce pools and rapids. On navigable streams, the shallow portions between pools are called "crossings."

Manning's Form. $V = \frac{1.486}{n} r^{\frac{2}{3}} S^{\frac{1}{2}}$

Chezy's Form $V = c\sqrt{rs}$

Critical Velocity Varies from (1'/sec Earth) to (10'/sec. rock)
p. 145

Swollen Stream Erodes

Sluggish Stream Silts.

CHAPTER VI

FLOW OF WATER IN TILE DRAINS

The general laws for the flow of water in tile drains are the same as those for open channels. In fact, pipes operate as open channels as long as the water flows through them solely by virtue of gravity, and not under pressure. The factors of slope, hydraulic radius, and roughness of surface govern the velocity in both cases. For pipes, however, the slope is much greater, the hydraulic radius much smaller, and the effect of roughness much less than for earth channels.

All of the early pipe formulas and some of the modern ones were developed for pipe either of individual lengths of 10 to 20 feet or of continuous construction, such as cast iron, steel, wood staves, and concrete. Only a few formulas have been designed especially for drain tile. The main difference between drain tile and other pipe, from the standpoint of flow, is the greater roughness of surface caused by the joints which occur 1, 2, or 3 feet apart, depending upon the diameter of the tile. The joints are not only more numerous but also rougher since it is difficult to prevent the individual tile from settling or moving slightly out of alignment, and since tiles are not truly circular and uniform in diameter. Another difference is that while other pipe serve simply as conductors of water introduced at one point to a more or less distant outlet, tile drains serve also as collectors of water, as water enters at each joint throughout the entire length of the drain.

Chezy-Kutter Formula.—Kutter's modification of Chezy's formula has been used more than any other for computing the capacity of tile drains, although it was developed for large open channels and not for pipes. Kutter's formula has been given and discussed in detail in Chapter V.

The slopes of tile drains are much larger than those for earth channels and are of such magnitude as not to affect the value of the coefficient; hence, the slope fraction can be omitted, if desired, in computations.

The hydraulic radius for a given diameter of tile is constant, because tile drains are designed for the tile flowing full, and is equal to one-fourth the diameter.

The roughness coefficient, n, is much less for tile than for earth channels and varies from 0.010 to 0.017. The values commonly used in designing are 0.013 and 0.015, though recent experiments show conclusively that a value between 0.011 and 0.012 is more nearly correct. In Figs. 70 and 71, a value of 0.011 is used in constructing the Kutter curves.

Manning's Formula.—This formula is repeated here for convenience.

$$V = \frac{1.486}{n} r^{\frac{2}{3}} s^{\frac{1}{2}} \tag{22}$$

If the value 0.011 is substituted for n, which is probably the correct one for tile drains, the formula becomes:

$$V = 135.1 r^{\frac{2}{3}} s^{\frac{1}{2}} \tag{23}$$

Although this formula has been used very little in the design of tile drains, it is undoubtedly the best of the older formulas, and the results obtained from it agree closely with the most recent experimental determinations as shown in Figs. 70 and 71.

The Poncelet Formula.—This formula was much used at one time, and recent experiments show that it has considerable merit for the smaller sizes of tile drains. The formula is:

$$V = 48\sqrt{\frac{dh}{l + 54d}} \tag{24}$$

in which,

V = mean velocity in feet per second.
d = diameter of tile in feet.
h = head, or difference in elevation in feet between the extremities of the drain which is considered.
l = length of drain in feet, corresponding to h.

This formula differs from the preceding ones in that the effect of slope is not constant for different lengths of drain. For example, if the length l is taken as 1,000 feet, the velocity is several per cent greater than if 500 feet are used, even though the slope is the same in each case.

Elliott's Formula.—G. C. Elliott in his book on "Engineering for Land Drainage" gives the following general formula:

$$V = m\sqrt{\frac{dh}{l + 54d}} \tag{25}$$

This differs from the Poncelet formula only in that the constant has been replaced by a variable coefficient. The formula applies to a well-laid, straight drain, running full on a uniform grade. The head, h, is assumed to be distributed so as to produce an even grade throughout the line.

The coefficient, m, varies with the size of the tile, and the values suggested by Elliott are given below:

Diameter of tile in inches	5	6	8	9	10	12	16	18	24	30	36	42	48
m	34	36	40	43	44	45	47	50	54	57	60	61	64

Elliott believed that in an open soil an additional head was produced by the water in the soil above the tile, and he proposed a modification of his general formula to meet this condition. His formula for open soils is,

$$V = m\sqrt{\frac{d(h + h')}{l + 54d}} \tag{26}$$

in which h' is a fractional part of the depth of the soil above the tile. In the formula as given in his book, referred to above, the value of h' is taken as one-half the depth of soil, though in the text he states that it will vary from 0.3 to 0.5, the depth depending on the openness of the soil. It is also stated that "the length of the drain to which the formula should be applied should be a representative part which is laid on the least grade."

A further modification was proposed by Elliott for the velocity of flow in a main drain having a number of submains flowing into it. The formula when thus modified becomes,

$$V = m\sqrt{\frac{d\left(h + \frac{b}{n}\right)}{l + 54d}} \tag{27}$$

The quantity $\left(h + \frac{b}{n}\right)$ in the numerator of the fraction represents the total head. The head of the main drain is h, the sum of the amounts in which the heads of the submains exceed that of the main is b, and the number of submains is n. The use of this formula "is restricted to mains which have submains not less than 6 inches in diameter and connected with that section of the main drain whose capacity is being computed."

The Williams-Hazen Formula.—This formula is:

$$V = C_w r^{0.63} s^{0.54} 0.001^{-0.04} \tag{28}$$

The value of C_w proposed by the authors of this formula for tile sewers is 110. This formula is used extensively in computing the capacity of water pipe and sewers, but has had little use in connection with tile drains. Nevertheless, it probably expresses the law of water flowing in tile drains more correctly than any other formula, and gives very satisfactory results if C_w is taken as 120. The principal objection to the formula is the difficulty in making computations from it. To eliminate this difficulty the authors have devised a special slide rule.

The U. S. D. A. Formula.—In 1916 and 1917, the drainage division of the Bureau of Public Roads, United States Department of Agriculture, conducted a series of experiments at Arlington, Virginia, on the flow of water in tile drains. The results were published in 1920 in *Bulletin* 854. The formula derived from these experiments is usually referred to as the U. S. D. A. formula.

This experimental investigation was conducted on a very comprehensive scale. The principal feature of the equipment was a wooden flume about 570 feet long, 2 feet wide, and 2 feet deep, in which the tiles were laid in earth exactly as in practice. The flume was constructed of 2-inch plank, and the joints and seams were calked with oakum. The earth in the bottom of the flume was about 7 inches deep and was placed in layers 2 inches deep and each layer thoroughly tamped. The tiles were laid on the bed thus prepared and blinded with fine earth, free from large clods, which was tamped on each side of the tile by the feet of the inspector in charge. After the tiles were covered in this way, the remaining space in the flume was filled with earth. The flume was adjustable to any grade up to 1.50 per cent without disturbing the tile. The depths of flow in the tile were observed by piezometer tubes hung on the sides of the flume. A constant flow of water through the tile line was maintained from a supply tank which received the discharge of an 8-inch centrifugal pump. The movement of the water from the discharge pipe was quieted by baffles. The discharge from the tank into the tile line was through a brass, triangular-notch weir, and the head on the weir was measured with a hook gage.

Experiments were made with all the commercial sizes of tile, both clay and concrete, from 4 to 12 inches inside diameter. Nine grades from 0.05 to 1.50 per cent were used for each size of tile. In all, 824 separate tests were made of which 143 were for the tile flowing from 95 to 100 per cent full, but not under pressure. From the results obtained, the following general formula was derived to apply to both clay and concrete tile and for all sizes from 4 to 12 inches.

U.S.D.A.

$$V = 138r^{\frac{2}{3}}s^{\frac{1}{2}} \tag{29}$$

It will be observed that the exponents of r and s are the same as those in Manning's formula and that the coefficients in the two formulas differ only by about 2 per cent for $n = 0.011$ in the Manning formula.

To determine the roughness coefficient for drain tile, Kutter's n was computed from the observed data for each of the 824 tests. A study of these values does not disclose any effect of either slope or size of tile. Table XXVI has been compiled from the data on n for the 143 tests in which the tile were flowing from 95 to 100 per cent full. Maximum, minimum, and average values of n are given for each size of both clay and concrete tile. The close agreement of the average values for each size of tile and the lack of any regular increase or decrease in n with changes in size show that the roughness coefficient is constant for all practical purposes for all sizes of tile.

Subsequent to the above experiments, measurements were made on the flow of water in a 30-inch concrete tile drain in Judicial Drain No. 15, Murray County, Minnesota, by C. E. Ramser,[1] Senior Drainage Engineer, Bureau of Public Roads. Measurements of the hydraulic slope and the depth of flow were made at two surface inlets, one 234 feet and the other 1,237 feet above the tile outlet. The grade of the tile line was 0.10 per cent. The value of n for the tile nearly full was found to be 0.01185, which is between the value of n for 10- and 12-inch concrete tile as obtained in the Arlington experiments. (Thus, it would appear that, for tile flowing full, the roughness coefficient is independent of the size of tile.)

(For tile flowing less than full, an increase in n was observed with a decrease in the depth of flow.)

[1] YARNELL, D. L., "The Flow of Water in Drain Tile," *Proc. Iowa Eng. Soc.*, p. 27, 1921.

The experimental data seem to disprove the commonly accepted theory that the velocity of flow in a pipe flowing one-half full is the same as in one flowing full.

TABLE XXVI.—KUTTER'S n FOR TILE DRAINS FLOWING FULL[1]

Diameter of tile in inches	Number of tests	Values of Kutter's n		
		Maximum	Minimum	Average
Clay Tile				
4	13	0.0114	0.0101	0.01091
5	17	0.0114	0.0102	0.01079
6	6	0.0111	0.0103	0.01057
8	15	0.0120	0.0112	0.01163
10	18	0.0114	0.0104	0.01095
12	4	0.0115	0.0108	0.01097
All sizes	73	0.0120	0.0101	0.01101[2]
Concrete Tile				
4	8	0.0117	0.0107	0.01125
5	9	0.0110	0.0103	0.01051
6	7	0.0116	0.0103	0.01093
8	17	0.0118	0.0108	0.01121
10	25	0.0125	0.0114	0.01188
12	4	0.0116	0.0110	0.01142
All sizes	70	0.0125	0.0103	0.01135[2]
Clay and Concrete Tile				
All sizes	143	0.0125	0.0101	0.01118[2]

[1] Compiled from *Bull.* 854, United States Department of Agriculture.
[2] Weighted average.

Since the average of the 143 values of n, as given in Table XXVI, is 0.01118, it follows that a value of 0.011 is the proper one to use in computing the capacity of tile drains by either the Kutter or the Manning formula.

Tables XXVII and XXVIII, which are reproduced from *Bulletin* 854, give a comparison of the velocities as determined experimentally and as computed from formula (29) and show how closely the formula represents the actual conditions of flow.

TABLE XXVII.—COMPARISON OF VELOCITIES FOR CLAY TILE FLOWING FULL

Bulletin 854, United States Department of Agriculture

Size of tile, inches	Grade, per cent	Velocity determined experimentally feet per second	Velocity by formula 29, feet per second	Difference between columns 3 and 4, feet per second	Size of tile, inches	Grade, per cent	Velocity determined experimentally feet per second	Velocity by formula 29, feet per second	Difference between columns 3 and 4, feet per second
(1)	(2)	(3)	(4)	(5)	(1)	(2)	(3)	(4)	(5)
4	0.05	0.60	0.59	−0.01	6	0.75	3.33	3.04	−0.29
	0.10	0.77	0.84	+0.07		1.00	3.78	3.51	−0.27
	0.20	1.08	1.18	+0.10					
	0.30	1.36	1.45	+0.09	8	0.05	0.87	0.95	+0.08
	0.50	1.76	1.87	+0.11		0.10	1.23	1.34	+0.11
	0.75	2.12	2.29	+0.17		0.20	1.82	1.89	+0.07
	1.00	2.44	2.64	+0.20		0.30	2.13	2.32	+0.19
	1.25	2.93	2.96	+0.03		0.50	2.92	2.99	+0.07
	1.50	3.27	3.24	−0.03		0.75	3.53	3.66	+0.13
5	0.05	0.60	0.68	+0.08	10	0.05	1.06	1.08	+0.02
	0.10	1.02	0.96	−0.06		0.10	1.57	1.52	−0.05
	0.20	1.35	1.36	+0.01		0.20	2.27	2.16	−0.11
	0.30	1.70	1.67	−0.03		0.30	2.74	2.65	−0.09
	0.50	2.11	2.15	+0.04		0.50	3.50	3.42	−0.08
	0.75	2.52	2.64	+0.12		0.75	4.23	4.19	−0.04
	1.00	2.83	3.04	+0.21		1.00	4.98	4.84	−0.14
	1.25	3.39	3.40	+0.01		1.25	5.63	5.40	−0.23
	1.50	3.90	3.73	−0.17					
					12	0.05	1.17	1.21	+0.04
6	0.05	0.77	0.79	+0.02		0.10	1.86	1.71	−0.15
	0.10	1.04	1.11	+0.07		0.20	2.35	2.41	+0.06
	0.20	1.63	1.56	−0.07		0.30	2.95	2.96	+0.01
	0.30	2.03	1.92	−0.11		0.50	3.94	3.82	−0.12
	0.50	2.59	2.48	−0.11					

Total plus difference	+2.11
Total minus difference	−2.16
Number of items	44
Average difference, foot per second	0.097

TABLE XXVIII.—COMPARISON OF VELOCITIES FOR CONCRETE TILE FLOWING FULL

Bulletin 854, United States Department of Agriculture

Size of tile, inches	Grade, per cent	Velocity determined experimentally feet per second	Velocity by formula 29, feet per second	Difference between columns 3 and 4, feet per second	Size of tile, inches	Grade, per cent	Velocity determined experimentally feet per second	Velocity by formula 29, feet per second	Difference between columns 3 and 4, feet per second
(1)	(2)	(3)	(4)	(5)	(1)	(2)	(3)	(4)	(5)
4	0.05	0.52	0.58	+0.06	8	0.05	0.92	0.90	−0.02
	0.10	0.78	0.82	+0.04		0.10	1.31	1.30	−0.01
	0.20	0.99	1.16	+0.17		0.20	1.89	1.84	−0.05
	0.30	1.18	1.41	+0.23		0.30	2.30	2.26	−0.04
	0.50	1.69	1.83	+0.14		0.50	2.99	2.91	−0.08
	0.75	2.15	2.24	+0.09		0.75	3.53	3.57	+0.04
	1.00	2.39	2.58	+0.19		1.00	3.98	4.11	+0.13
	1.25	2.82	2.89	+0.07		1.25	4.42	4.61	+0.19
	1.50	3.06	3.16	+0.10		1.50	4.83	5.04	+0.21
5	0.05	0.61	0.67	+0.06	10	0.05	0.90	1.07	+0.17
	0.10	0.97	0.95	−0.02		0.10	1.38	1.52	+0.14
	0.20	1.36	1.32	−0.04		0.20	2.06	2.15	+0.09
	0.30	1.71	1.65	−0.06		0.30	2.47	2.63	+0.16
	0.50	2.30	2.13	−0.17		0.50	3.23	3.39	+0.16
	0.75	2.69	2.61	−0.08		0.75	3.87	4.16	+0.29
	1.00	3.19	3.01	−0.18		1.00	4.49	4.80	+0.31
	1.25	3.53	3.37	−0.16		1.25	4.99	5.37	+0.38
						1.50	5.37	5.88	+0.51
6	0.05	0.67	0.76	+0.09					
	0.10	1.13	1.08	−0.05	12	0.05	1.20	1.21	+0.01
	0.20	1.58	1.53	−0.05		0.10	1.72	1.71	−0.01
	0.30	1.97	1.87	−0.10		0.20	2.37	2.42	+0.05
	0.50	2.30	2.41	+0.11		0.30	2.93	2.97	+0.04
	0.75	2.95	2.95	+0.00					
	1.00	3.26	3.41	+0.15					
	1.25	3.77	3.81	+0.04					
	1.50	4.28	4.18	−0.10					

Total plus difference	+4.42
Total minus difference	−1.22
Number of items	48
Average difference, foot per second	0.117

Comparison of Formulas.—In order that the several formulas, which have been given, may be more readily compared, Figs. 70 and 71 have been prepared; the former for 8-inch tile, and the latter for 30-inch tile.

It will be observed that for the 8-inch tile Kutter's formula gives velocities which are from 0.10 to 0.32 feet per second smaller than those given by the U. S. D. A. formula, although a value of n equal to 0.011, which was obtained from the Arlington experiments, is used in Kutter's formula. For the 30-inch tile, the reverse is true, the Kutter velocities being greater than those given by the U. S. D. A. formula.

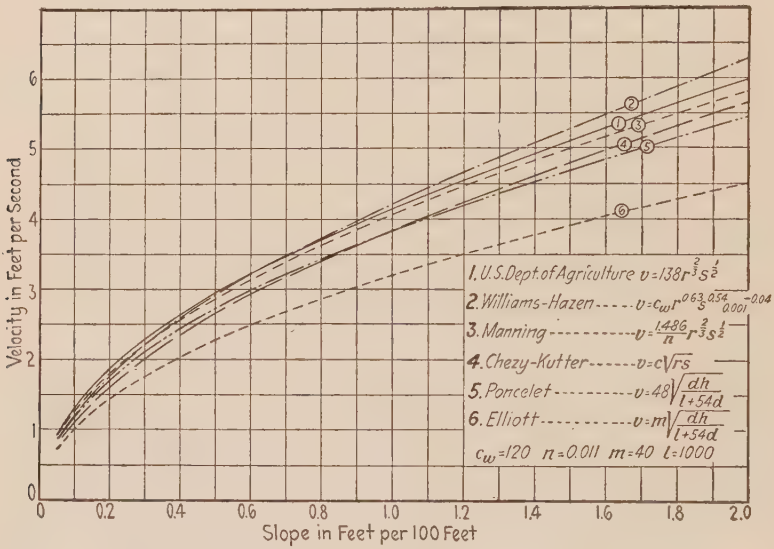


FIG. 70.—Graphical comparison of formulas for flow of water in 8-inch tile drains.

As would be expected from the similarity in formulas, the velocity curves for Manning's formula and for the U. S. D. A. formula agree closely for both sizes of tile.

The Williams-Hazen curve also agrees closely with the U. S. D. A. curve for the 8-inch tile, but gives smaller values for the 30-inch tile. It will be noted that while the Williams-Hazen curve gives smaller velocities, as compared with the U. S. D. A., as the size of tile increases, the Kutter curve gives larger values.

The agreement of Poncelet's curve and Kutter's curve for the 8-inch tile is also to be noted. For the larger sizes, however, Poncelet's curve departs widely from the others.

When the extensive use of Elliott's formula and the satisfactory results which it has given in practice are considered, the wide departure of Elliott's curve from the others is rather sur-

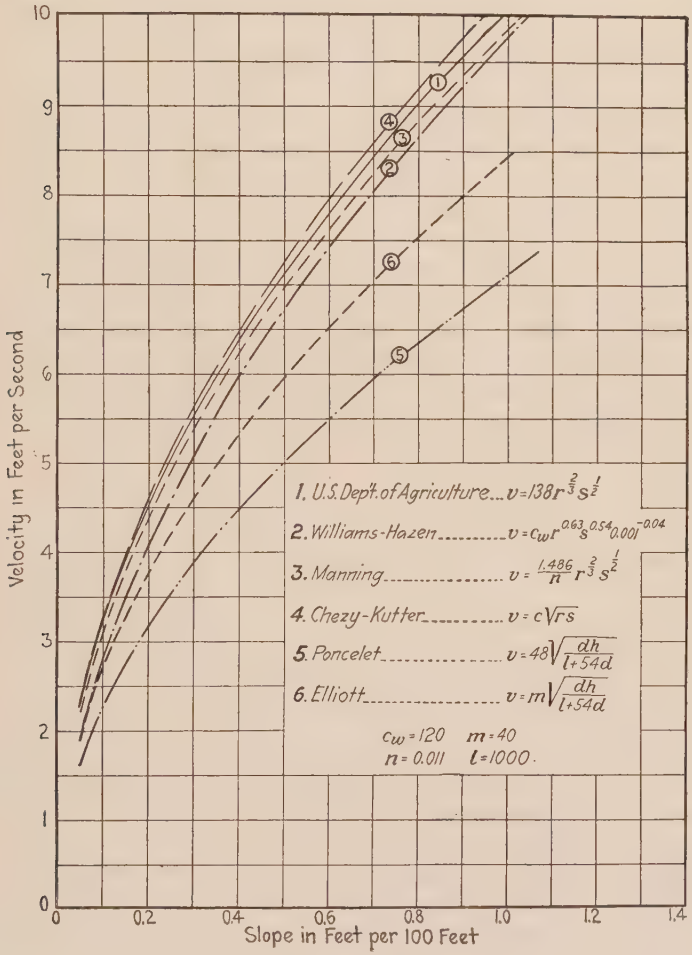


FIG. 71.—Graphical comparison of formulas for flow of water in 30-inch tile drains.

prising. It is to be remembered, however, that there are two uncertain factors which enter into the satisfactory performance of a tile-drain system. One is the carrying capacity of the drain and the other is the drainage modulus or the amount of water to

be removed. The satisfactory results which have been obtained with Elliott's formula are due no doubt to the underestimation of the drainage coefficient which has offset the smaller carrying capacity of the tile as computed from the formula. Elliott is the only author who has considered the effect of the pressure of the water in the soil above the tile upon the velocity of flow in the tile, formula (26). As pointed out by Elliott, tile drains laid on very flat grades in open soil under conditions of maximum flow discharge a greater volume than that which would be caused by the head provided by the slope; but this additional pressure is probably caused by the water in the tile laterals rather than by the water in the soil. If the water reaches the laterals faster than they can remove it, the tile will flow full and exert a pressure on the water in the tile main. If the drains are large enough to remove the water as fast as it reaches them, the water in the soil can never exert a pressure on the water in the tile. This factor, therefore, should not be introduced into the velocity formula, but rather should be considered as an additional factor of safety.

Retarding Influences.—With the exception of the U. S. D. A. and Elliott's formulas, all the formulas which have been given were developed for pipe lines other than tile drains, and the only retarding influence considered is the roughness of the inside surface of the pipe. The U. S. D. A. formula is the only one which is based on experiments made on tile drains, and is the only one which includes the retarding influence of the more or less uneven joints between the individual tiles. There are other retarding influences, however, which are peculiar to tile drains; and although these influences have not been studied experimentally and their magnitudes evaluated, they should not be ignored in designing a tile-drain system.

Effect on Flow of Water Entering Joints.—A tile-drain system is composed of three classes of tile lines, based on the condition of flow within them: (1) the laterals; (2) the submains; and (3) the mains.

The laterals are comparatively short, parallel lines of 4- or 5-inch tile, spaced from 50 to 200 feet apart, and generally are of uniform size throughout their lengths. In the average system, the combined lengths of laterals represent about 90 per cent of the total length of all the tile lines, and are, therefore, a very important part of the system. Let us consider the condition of flow within

a tile lateral. As previously stated, the ground water enters the drain at the joints. The water which enters the first joint at the upper end of a lateral is very small—probably about 2 per cent of the capacity of the drain—and hence the hydraulic grade line may be assumed to start at the elevation of the invert of the drain. At the succeeding joints approximately equal inflows will enter the drain, since each joint theoretically serves a strip of land 1 foot wide and from 25 to 100 feet long on each side of the drain; but these inflows will be less than that at the first joint, since it serves a much wider strip above the end of the lateral. The water entering the joints must have a greater or less retarding effect upon the flow of the water in the tile, although this effect decreases in magnitude as the volume and velocity of flow increase as the distance from the upper end of the lateral increases. It is impossible to say what the magnitude of this effect is. (Fortunately, it plays little or no part in the design,) since the laterals seldom, if ever, flow full except possibly at their lower ends. The flow in laterals may be visualized as increasing in depth from zero at the upper end to a possible maximum equal to the diameter of the tile at the lower end, and as increasing in velocity from zero at the upper end to a maximum —depending upon the diameter of drain and the slope—at the lower end.

The size of laterals, however, is not determined altogether according to their capacities, as is the case with submains and mains. Laterals are universally constructed of 4- or 5-inch tile, and these sizes are in most instances much larger than required to collect the ground water, provided the tiles do not get out of alignment nor become partially obstructed by the deposition of silt which is carried into them by the ground water, especially during construction and the first year after construction.

The effect of the water entering the joints upon the flow in submains and mains is very slight. In the first place, not as much water enters the joints as in the case of laterals, because the land through which the submains and mains pass is drained in large part by the laterals. Further, these drains carry a greater volume of water than the individual laterals, and hence the disturbance of flow caused by the ground water entering the joints is less.

Flow of Water in Submains.—The flow in a submain is derived almost entirely from the discharges of the laterals, which enter

the submain at intervals of from 40 to 150 feet, depending upon the design of the system; hence, the volume of flow in a submain increases from the upper to the lower end, which means that the hydraulic grade line is somewhat less than that of the invert of the drain, and that the discharge is less than the computed discharge, based on the slope of the invert. The velocity and discharge are further affected by the successive enlargements in the diameter of the submain; that is, the upper end of a submain may be 5 inches in diameter and the lower end 8 or 10 inches in diameter, the drain being enlarged as required to carry the gradually increasing discharge. Thus, at the upper end of the 5-inch stretch of drain, the drain never flows full, but the depth of flow increases as each lateral adds its discharge; and at the end of this stretch the tile will flow approximately full. At this point the size of the drain is increased to 6 inches, which means that the 6-inch tile is only about two-thirds full; but the depth of flow in the 6-inch tile increases with each increment of discharge from the laterals until it is flowing approximately full. At this point the size of drain is increased to 8 inches, and the discharge is carried at about 0.6 depth in the 8-inch tile. This process is continued to the end of the submain. The flow of the water is retarded to some extent by the sudden enlargements in the size of the drain, although this loss of head is largely offset by the vertical drop in the grade line equal to one-half the difference in the diameters of the two connecting drains.

(The greatest retarding influence in a submain is the disturbance in the flow caused by the discharges of the laterals.) If the laterals enter the submain at a right angle, the retardation of flow is considerable; and for this reason, the laterals in a properly designed system are connected to the submain at a small angle and in such a manner that the surfaces of the two streams of water will be at about the same elevation under conditions of maximum flow. (The retarding effect of the impact of the two streams of water has never been investigated experimentally, and hence our knowledge of this factor is incomplete. In design, however, it is allowed for by increasing the size of the submain shortly before the drain would flow full according to the theoretical formulas.)

The main drain simply discharges the water brought to it by the submains, which are comparatively few in number, and, therefore, the conditions of flow are more constant than in the

other drains of the system. The formulas given express the flow in mains sufficiently accurate for all practical purposes.

Curves.—The retardation of flow due to curves in the drain is negligible except in the cases of very sharp ones. Experiments

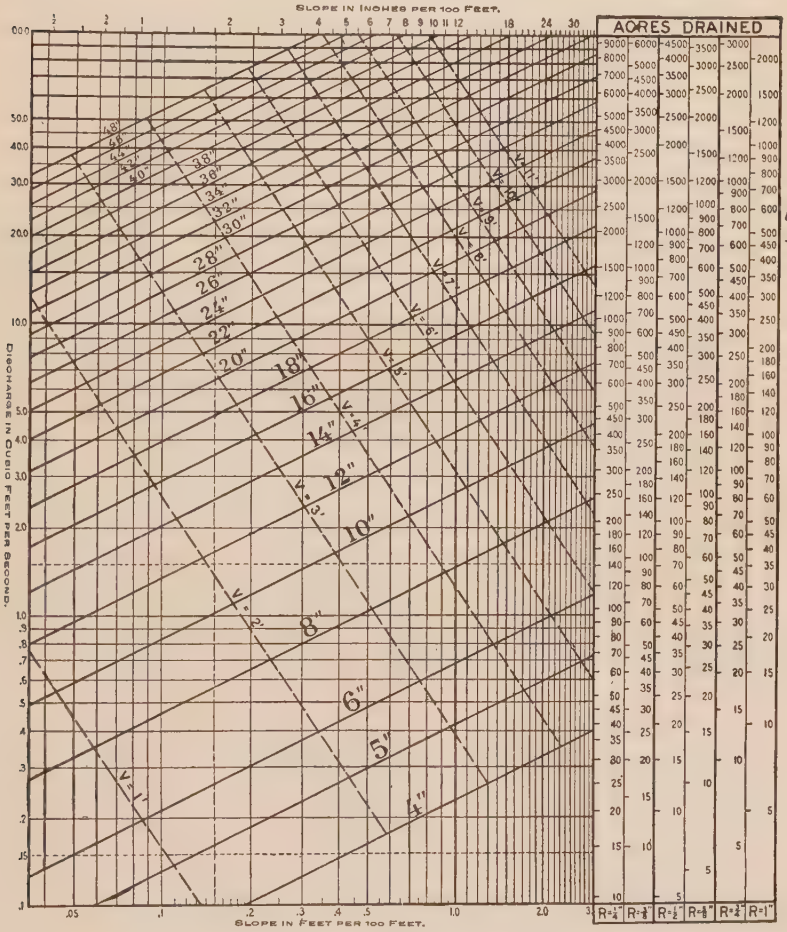


FIG. 72.—Discharge diagram for tile drains, based on U. S. D. A. formula, $V = 138R^{\frac{2}{3}}S^{\frac{1}{2}}$.

indicate that if the radius of curvature be as much as five times the diameter of the pipe, no appreciable loss of head will result. It is always possible in constructing tile drains to meet this condition.

Obstructions.—Naturally, obstructions which form in a drain will affect the flow. The more common obstructions which occur in tile drains are deposits of fine sand carried into the tile through the joints, and roots of trees which enter the drain in search of water. Large drains are seldom obstructed in this way. For such drains, the chief source of obstruction is the breaking of defective tile under the pressure of the ditch-filling material. There is no reliable information as to the minimum velocity which will prevent silting in tile drains, but the general opinion is that this velocity is about 1.5 feet per second.

These are factors which cannot be allowed for in the design, except in the case of laterals, where a reasonable amount of silting is provided for. It is in the construction of tile drains that the occurrence of obstructions can be made a minimum by careful workmanship and the use of the best materials.

Conclusions.—The author recommends the use of either the U. S. D. A. formula or the Manning formula. For the Manning formula, values of n of 0.011, 0.012, and 0.013 are recommended for use in designing mains, submains, and laterals, respectively. It is believed that the U. S. D. A. formula gives velocities which are too large for submains and laterals, since the Arlington experiments did not take into account some of the factors which tend to decrease the velocity in such drains. It is recommended that the values obtained from this formula be reduced 10 per cent for submains and 15 per cent for laterals. The diagram in Fig. 72 is reproduced from *Bulletin* 854 to simplify the use of the U. S. D. A. formula.

CHAPTER VII

LAND DRAINAGE BY OPEN CHANNELS

The ultimate purpose of artificial drainage is to increase the productivity of agricultural soils by removing the free water which is injurious to plant growth. As pointed out in the chapter on Soil Physics, the ordinary crops will not grow in a saturated soil, nor will they thrive in one in which more than about 50 per cent of the voids are filled with water.

In rolling country, the surface slopes are sufficient to shed the precipitation in excess of that which can be absorbed readily by the soil, and numerous natural swales and depressions carry it away to the creeks and rivers, thereby keeping the water table well below the surface. Over flat areas, on the other hand, no such natural drainage exists, and the precipitation saturates the soil and even collects in shallow depressions on the surface, until it either evaporates or moves slowly through the subsoil to a more or less distant stream. This condition exists on the flat uplands of the prairie states, in the bottom lands along alluvial streams, and in the swamps adjacent thereto where water stands on the surface the year round. It is over such areas that artificial drainage is needed.

The drainage works necessary to drain these areas satisfactorily may be considered under two classifications: (1) a system of tile underdrains which collect the excess water which percolates through the soil; and (2) a system of open channels which serve as outlets for the tile-drain systems, and which collect the surface flow during excessive precipitation and discharge it into some natural channel capable of receiving it. This chapter treats of the second classification; the first classification is considered in a subsequent chapter.

Preliminary Investigations.—One of the first investigations to be made by the engineer is in regard to the fertility of the soil of the area whose drainage is contemplated, since upon it will depend the expenditure which is justified in the reclamation of the land. The fact must be recognized that the drainage of wet

land is primarily a business enterprise, and that if the increased income from the land, as a result of the improvement, is not greater than the interest on the capital invested in the drainage system, the undertaking is a failure, regardless of the efficiency of the drainage system itself. The improvement must be paid for eventually from the wealth which is stored in the soil, and the amount of this wealth is represented by the soil's fertility. An invoice of the soil is just as essential in drainage enterprises as is an investigation into the financial returns to be derived from the construction of other engineering projects. This invoice tells how much wealth there is in the soil to draw on. Each crop takes a certain amount of fertility out of the soil, and, unless the plant food exists in abundance or unless those elements taken out are put back by proper farming methods, such as rotation of crops, or by fertilizing, the soil will soon lose its productiveness.

The fact that the owners of the land desire to drain it is usually sufficient evidence of the economic soundness of the enterprise, though this is not invariably true. Where crops have been raised on the land during dry years, the yields obtained may prove conclusively that the soil is highly fertile; but where the areas have never been cultivated, this proof is lacking. Most river-bottom and swamp lands are extremely fertile, but not all such lands are so, and the owners of the land may be mistaken in their conclusions. Or it may be that the land has been bought by a development company whose chief aim is to drain the land and to sell it at as large a profit as possible, leaving the purchasers to bear the loss in case the land is not productive.

Unless the fertility of the soil is beyond question, the engineer should secure representative samples of the surface soil and subsoil and have them analyzed, thus obtaining the amount of the various elements of plant food per acre. The cost of such an analysis is small, and a definite knowledge of actual conditions is worth much both to the engineer and to the landowner.

Several states through their agricultural colleges are conducting soil surveys and publishing soil maps which give the required information. If such maps are available for the area under consideration, they should be obtained and carefully studied.

Economics of Land Drainage.—The fertility of the soil having been determined, the engineer has a sound basis upon which to estimate the maximum expenditure which the proposed

improvement will warrant. The cardinal principle of land drainage is that the benefits to be derived must exceed the cost of the drainage works.

A conservative estimate of the increased gross revenue which the land will produce per acre, as the result of drainage, should be made after consultation with the landowners and others whose opinions have value. The results of drainage on some near-by district of similar soil conditions is probably the best criterion.

The net income is found by deducting from the gross income the cost of raising a crop. This cost must include:

1. The interest on original investment in wet land, and taxes based on the new valuation of the land.

2. Interest, taxes, insurance, maintenance, and depreciation on any buildings which must be constructed in order to farm the land. These items will amount to about 12 per cent of the cost of the buildings. Since the cost of buildings required for a small farm varies little from that for a large one, the cost per acre may vary considerably even in the same drainage district. In this case, the average size of farm is taken as a basis. Where the area to be drained is part of farms which are already equipped with buildings, additional ones may not be necessary, and the building charge can be omitted.

3. Interest, maintenance, and depreciation on the farming machinery and equipment necessary to farm the land. These items will amount to about 14 per cent of the cost of the equipment. The cost per acre of this charge will also vary with the size of the individual farms, though to a much smaller extent than that in item 2.

4. Annual cost of the seed, and the cost of preparing the ground, planting the seed, cultivating the growing plants, harvesting the crop, threshing, hauling to elevator, etc. This cost depends mainly on local condition of labor, but it is readily estimated for a given locality.

The net increase per acre thus obtained represents the monetary benefit to the landowner, and it is from this sum that the drainage assessment for construction and maintenance should be paid. The fact that several annual assessments may have to be paid before the land is producing revenue must also be recognized. Theoretically, if all this sum were used in paying the annual drainage assessment, the landowner would still be benefited since he would receive interest on all capital invested and

wages for his own labor, in addition to his gradually increasing equity in the drainage works. The landowner, however, is seldom willing to spend all the benefits, and it is rarely necessary that he should. Generally, the benefits derived greatly exceed the cost of drainage.

It must be remembered that an open ditch system alone will not provide complete drainage, and the cost of the ditches must not be so much that, when the cost of tile drainage is added, the total cost will exceed the benefits.

The estimate of benefits assists the engineer to solve another problem, namely, the degree of protection which the drainage system should give against flooding and loss of crops. The more productive the land, the greater is the protection which is warranted. The gross value of a crop (exclusive of the cost of harvesting) will determine whether the ditches shall be large enough to remove satisfactorily the run-off from the maximum precipitation which occurs with a frequency of once in 10 years or that which occurs once in 25 years or more. This is purely an economic problem after a frequency curve of run-off has been decided upon as a result of rainfall and run-off studies. By trial the engineer determines the point where the cost of additional protection is just equal to the value of that protection. Generally, it is more economical to lose a crop once in 15 or 20 years than it is to provide protection against the maximum flood which, on the average, will occur in that length of time.

Physical Data.—All of the physical features of the area to be drained, which will have a bearing on the design of the drainage system, should be obtained by a survey and plotted to a convenient scale. The principal data to be shown are: (1) the boundaries of the watershed; (2) all possible outlets for the drainage system; (3) all existing watercourses, sloughs, and bayous which might be used as part of the system, together with profiles and cross-sections of the same; (4) highways and railroads; (5) location, size, and condition of all bridges; (6) villages and isolated buildings; (7) property lines and section lines; (8) areas under cultivation; (9) swamps and wooded areas; and (10) the relative elevations of all parts of the watershed, which may be shown either by 1-foot contours or by numerous elevations marked on the map.

In making drainage surveys, the vertical control is of much greater importance than the horizontal control, and special

care should be observed in running and checking the lines of control levels and in determining the elevations of all low spots, since an error in this work may vitally affect the success of the project. Surveys of swamp areas are the most unpleasant and most difficult to make on account of the nature of the surface. Winter is the best time to make surveys of these areas, when the ground and standing water are frozen. Highways and railways are generally used for the horizontal control. Starting where these lines intersect section and quarter-section lines, levels are run along these lines with level and stadia, thus giving a series of parallel profiles across the swamp area.

In addition to the above data, borings should be made to a depth of 12 to 15 feet to determine the nature of the subsoil, since the depth of the channels will be affected by the stability of the material through which they are to be constructed.

Location of Outlets.—The first step in the design of any drainage system is the location of the outlet. In most cases the outlet is apparent, and is the river or creek which drains the watershed; but occasionally the land to be drained is located on the level area at the head of, and common to, two or more watersheds, and an outlet may be had in two or more directions without violating the laws of most states which prohibit the drainage of water from one natural watershed to another through the divide which separates them.

Where several outlet streams are available, the one which provides the most efficient outlet should be chosen, unless the additional length of ditch required makes the cost prohibitive. It may be necessary to make preliminary designs and estimates for two or more systems, each utilizing one of the available outlets, before the best outlet can be determined. Or it may be that more than one outlet will be used, and part of the area drained in one direction and part in another.

When there is no question as to the watershed in which the land to be drained lies, and as to the stream to be used for an outlet, the point at which the main ditch of the drainage system should enter the outlet stream may be in doubt, and a careful study is necessary to determine the best place. This condition is likely to occur in very flat areas where there are no definite lines of surface drainage.

The outlet stream itself must be studied to determine its efficiency as an outlet. It may be that the channel is already

taxed to its full capacity during flood periods, and that it will be unable to dispose satisfactorily of the additional discharge to be brought to it by the proposed system. This may require the enlargement of the outlet stream for a greater or less distance below the end of the drainage system. This, of course, means a greater flooding of the lands downstream; but under existing state laws, the owners of the higher lands have this right, and a legally organized drainage district can go beyond the corporate limits of the district to secure an adequate outlet. This will be necessary in all cases where the main ditch of the drainage system is simply an enlargement of an existing natural channel. Here the outlet stream is simply a continuation of the main ditch, and were it not improved below the end of the district, the lands at the lower end of the district would be damaged instead of benefited.

Thus, it is evident that the successful drainage of all the lands in a given area depends upon the efficiency of the outlet, and hence its location must be given the consideration which its importance warrants.

Location of Main Channels.—After the outlet has been decided upon, the next step to consider is the location of the main ditch and its principal branches, which form the "backbone" of the drainage system. The former ditches are generally referred to as "mains" and the latter as "submains."

Since these channels serve as outlets for all parts of the watershed, they must obviously occupy the lowest ground. The contours or elevations plotted on the map will usually indicate the location of the ditches approximately, and in many cases within narrow limits.

The main ditch will start at the chosen outlet and extend upslope to the upper end of the watershed, following the most centrally located natural depressions. The submains will branch from the main and follow the secondary lines of natural drainage. Sometimes the source of the main ditch is the junction of two important submains. Mains and submains serve the same function, and differ only in length and size of channel.

When the land is flat and devoid of any marked natural depressions, the engineer has wider latitude in the location of the ditches. In this case, the main ditch is located approximately on the major axis of the drainage area, and as far as possible should follow a section or half-section line, since such lines are usually the dividing lines between landowners. The submains also

follow land lines at right angles to the main ditch and are located on each side of the main at wide intervals. For narrow areas, submains will not be needed and the lateral ditches, which serve as the direct outlets for the tile drains, enter the main directly. For still narrower areas, the laterals are unnecessary, their function being served by large tile drains.

For extremely large areas, two or more mains may be used, each with an independent outlet. This is equivalent to dividing the large area into two or more parts and designing a separate system for each part.

When the area to be drained is long and narrow, as is the case of bottom lands along small creeks, the main ditch will follow the general course of the existing stream, the straighter portions of the old channel being used and the crooked portions eliminated by cut-offs. The new channel should be made as straight as practicable in order to obtain a maximum slope.

Location of Laterals.—The land within 2,000 or 3,000 feet of the main or submain can be drained directly into them through a system of tile drains; but the land farther away cannot be drained satisfactorily in this way. For large areas, therefore, numerous small ditches called "laterals" are necessary to collect the water from the tile drains and from the surface and convey it to the larger channels. Such laterals are generally straight—except possibly at their lower ends—and are located along land lines at intervals of ½ or 1 mile. Frequently, the 1-mile interval is used in the original plans with the intention of adding intermediate laterals, if they are found to be necessary, after the land has been cleared and placed under cultivation. Such a system of laterals places even the most remote areas within from ¼ to ½ mile of an adequate outlet, which is the purpose of the ditch system.

It must be clearly understood that an open channel system, such as has been described, will not provide thorough drainage for the land—except for the areas immediately adjacent to the ditches—and that it must be supplemented by a system of tile drains to remove quickly the water which percolates through the soil.

Floodways.—In the reclamation of river-bottom land there are two distinct problems to solve: First, the protection of the land from overflow from the river and from hill streams; and second, the removal of the water which falls directly on the land as rain. If the stream is small and sluggish, the channel can be enlarged

and straightened so as to carry the flood discharge within its banks. It usually happens, however, that the cost of a channel wide enough to do this is prohibitive, and minor channel improvement must be supplemented by levees on one or both sides, depending upon whether the land to be reclaimed is on one or both sides of the stream. The levees act as artificial banks and the area between them is called a "floodway." Similar floodways may be necessary to divert large tributaries around the district to be drained or to carry their flood waters through the district. Since the level of the water in the floodway during flood periods is higher than the adjoining land, the floodway cannot be used as a gravity outlet until after the flood waters have subsided; hence, the main ditches generally parallel the floodway and connect with it at, or below, the end of the district. When a floodway is necessary, its location should be determined before the locations of the drainage ditches are decided upon. The design of floodways is considered in the chapter on flood control by levees, and the subject is mentioned here to call attention to the fact that frequently land drainage and flood control are intimately related.

Alignment of Ditches.—Since the function of drainage ditches is to remove quickly the water brought to them during storm periods, and thus prevent the flooding of the land, it is obvious that they should be as direct and as straight as possible. Generally the laterals are straight, but the mains and submains are rarely so, since they follow more or less closely the lines of natural drainage.

Where a change of direction is necessary, the tangents are joined by a curve. The radius of curvature which should be used depends upon the velocity of the water, which is determined by the size and slope of channel, and upon the stability of the earth. As far as possible the curvature should be adjusted to these two factors so that serious erosion of the outer bank will be avoided. Curves from 4 to 20 degrees are used in drainage work; the former for ditches with large discharges or with steep slopes, and the latter for ditches with small flow or with comparatively flat slopes. The velocity at bankful stage is usually the deciding factor and, where such velocities are high, as flat a curve as is consistent with local conditions should be used.

Erosion is more likely to occur at the ends of a curve than at the center, and this tendency can be reduced by the introduction

of a short easement curve at the ends of the circular curve. Figure 73 illustrates such a design. The beginning and the end of the curve—points A in figure—are located by measuring the tangent distances, T, from the point of intersection of the two tangents. The tangent distances are computed from the formula,

$$T = M \tan \frac{I}{2} + N \qquad (30)$$

The angle I is measured in the field and the variables M and N are given in columns 3 and 4 of Table XXIX. The curve

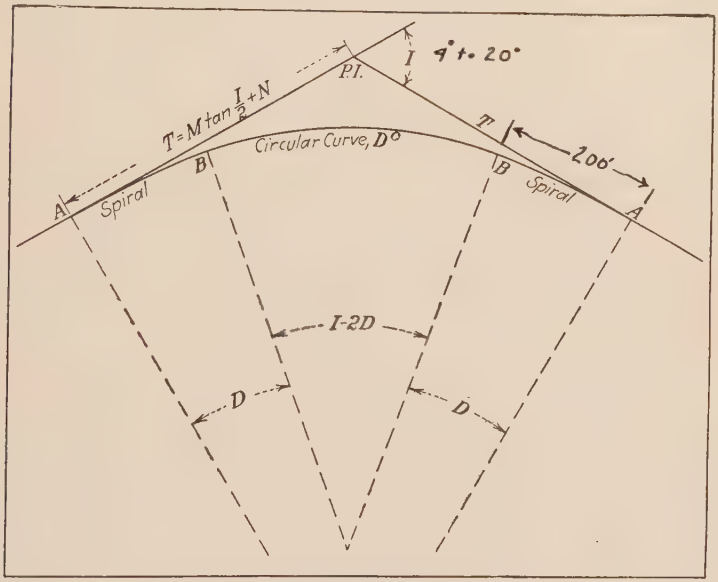


Fig. 73.—Curve recommended for open ditches.

AB is 200 feet long in every case. It is run-in best by the deflection-angle method. With the transit on tangent at A, the 50-, 100-, 150-, and 200-foot points on the easement curve are located by turning off from the tangent the deflection angles given in columns 5, 6, 7, and 8, respectively. After point B is set, the instrument is moved to this point and oriented by backsighting on the point A with the telescope inverted and the vernier reading the angle given in column 9. When the telescope is plunged and the vernier set at zero degrees, the line of sight will be on tangent at point B. (The circular curve is then run in in the usual manner by turning off successive deflection angles equal to one-

fourth the degree of curve for 50-foot chords until the point B on the forward end of the curve is reached.) The length of the circular curve is obtained from the formula,

$$L \text{ (in feet)} = \frac{(I - 2D)100}{D} \tag{31}$$

TABLE XXIX.—CURVE DATA

Degree of curve D	Radius of curve R	M	N	Deflection angles at A				Orientation angle at B
				A-1 (50 feet)	A-2 (100 feet)	A-3 (150 feet)	A-B (200 feet)	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
4	1,432.7	1,433.9	100.0	0–05	0–20	0–45	1–20	2–40
5	1,146.3	1,147.7	100.0	0–06	0–25	0–56	1–40	3–20
6	955.4	957.1	100.0	0–07	0–30	1–08	2–00	4–00
7	818.6	820.7	100.0	0–09	0–35	1–19	2–20	4–40
8	716.3	718.7	99.9	0–10	0–40	1–30	2–40	5–20
9	636.8	639.4	99.9	0–11	0–45	1–41	3–00	6–00
10	573.1	576.0	99.9	0–12	0–50	1–52	3–20	6–40
11	521.1	524.3	99.9	0–14	0–55	2–04	3–40	7–20
12	477.7	480.8	99.9	0–15	1–00	2–15	4–00	8–00
13	441.0	444.8	99.8	0–16	1–05	2–26	4–20	8–40
14	409.3	413.4	99.8	0–17	1–10	2–37	4–40	9–20
15	382.0	386.4	99.8	0–19	1–15	2–49	5–00	10–00
16	358.2	362.8	99.7	0–20	1–20	3–00	5–20	10–40
17	337.1	342.0	99.7	0–21	1–25	3–11	5–40	11–20
18	318.4	323.6	99.7	0–22	1–30	3–22	6–00	12–00
19	301.6	307.2	99.6	0–24	1–35	3–34	6–20	12–40
20	286.6	292.4	99.6	0–25	1–40	3–45	6–40	13–20

Since the arc BB is rarely a multiple of 50, there will be a subchord at the end of the circular curve. The subdeflection angle corresponding to this subchord of length c is:

$$d \text{ (in minutes)} = 0.3cD \tag{32}$$

The second spiral is located in the same manner as the first with the instrument at A.

Discharge.—The maximum discharge for which each ditch of the drainage system is to be designed must next be determined. The area contributing to the discharge of each ditch at as many

points as desired is obtained from the map. These areas multiplied by the drainage modulus give the probable maximum discharges at the various points. The drainage modulus is the depth of water in inches which must be removed from the drainage area in 24 hours. This factor has been treated fully in the chapter on run-off, pages 81–83, and hence will not be discussed here, further than to say that it is the most uncertain and the most important factor entering into the successful design and operation of a drainage system.

Grades.—After the system has been located on the map, profiles of the ground surface along the center line of each ditch are drawn from the data on the map. The engineer usually has little choice in establishing the grade lines, since these are determined almost entirely by local topographic and soil conditions. The grade of a ditch must follow more or less closely the slope of the ground surface in the direction of the ditch lines.

The elevation of the bed of the outlet stream, either in its natural state or as lowered by dredging, gives a starting point for the grade line of the main ditch. It may be necessary occasionally, however, to make the bottom of the ditch lower than the bed of the outlet. This occurs where the outlet of the drainage system is a broad, shallow stream or a swamp. In this way the hydraulic radius and the discharge are increased. To obtain this increased discharge, the width of the channel at the lower end must be increased to compensate for the decreased depth at the outlet. The raised outlet acts as a dam and impounds the water during the season of low flow, and this may be objectionable because of the stagnant condition of the water at that time.

The grade of the main ditch should be as uniform as possible throughout, and as large as is consistent with the function of the ditch, which is to provide adequate outlets for the submains and the laterals. The stability of the soil against erosion is a factor which must be considered in some instances. First, a tentative grade is assumed for the main. This gives a starting point for the grade line of each submain, and tentative grades are established which form the basis of the grades of the laterals. As the study progresses, the tentative grades will have to be modified somewhat in order to meet more satisfactorily the requirements of the other ditches. Even after such changes have been made, further adjustments may be necessary later when the sizes of channels and the velocity of flow are considered.

The slopes used for ditches range from 0.00005 to 0.0012, or from 3 inches to 6 feet per mile. Occasionally, slopes greater than 6 feet per mile are used. The minimum slope is necessary only when the ditch is very long and the area drained is very flat. This implies a large channel cross-section which compensates for the smaller slope. The maximum grade applies to short ditches of small section and to areas of fair natural slopes. Since the terms main, submain, and lateral are relative and do not indicate the size of the ditch, it is impracticable to specify ranges of slopes applicable to each. A lateral in a large system may be larger than the main ditch of a small system and may have a flatter slope. For a particular system, however, the main ditch will usually have the flattest slope, and the laterals the steepest slopes.

The nature of the soil through which a ditch is to be constructed also has a bearing on the establishment of a grade line since it may affect the depth of the ditch. For instance, sand may be encountered at a depth of 8 or 10 feet which will make a shallower ditch advisable than would be the case were the soil a clay. The borings made during the survey, or at a later period, will give the necessary data on subsoil conditions.

Slope is only one of the hydraulic elements entering into the design of a ditch, but since its value is fixed within narrow limits by topographic conditions, while the other elements are independent of such conditions, it is the first to be considered, although it is not definitely determined until all the factors have been studied.

Depth of Channel.—The depth of channel is one of the factors considered in establishing the grade line. The depth must be sufficient at all points to drain the area satisfactorily, and at the same time enable such slopes to be used as will result in an economic size of channel. An additional depth of 1 foot or more should be provided to take care of the silting which is more pronounced during the first few years after construction than after that period. Where these conditions can be met with some leeway, the soil conditions enter the problem. As previously pointed out, a sandy subsoil may make a comparatively shallow ditch advisable. In muck and peat soils, deeper channels are necessary than in other soils to allow for the subsidence which occurs when the soil is drained and cultivated. In the virgin muck soils of the Florida Everglades, it has been found necessary to make

the laterals 8 feet deep in order to have a depth of 5½ feet 3 or 4 years after construction. Generally, the material encountered is clay and the depth is controlled by factors other than the nature of the soil.

The usual depth of drainage ditches is from 6 to 12 feet; the former for laterals, and the latter for mains. The type of excavator most likely to be used in the construction of the ditch should also be considered, since certain types and sizes of machines cannot be used economically or satisfactorily if the depth is excessive.

The maintenance of the system, as well as the first cost, may affect the choice of depth. Silt is deposited more readily in shallow ditches than in deep ones, and weeds and grasses grow more abundantly, thereby reducing the efficiency of the system.

Shape of Channel Cross-section.—From the velocity formula, $V = c\sqrt{rs}$, it is evident that the velocity will increase as the hydraulic radius increases. For a given area and slope, that cross-section which has the smallest wetted perimeter will have the largest hydraulic radius and will give the greatest velocity and discharge. The semicircle is the form of cross-section which gives the maximum hydraulic efficiency, since for a given area its wetted perimeter is less than that of any other form. A semicircular cross-section, however, is impracticable for earth channels and is not used. Next to it in order of efficiency is the trapezoidal, and the latter is the standard form of cross-section for open channels. Of the many trapezoidal forms possible, the half hexagon is the best. In this form of section. the hydraulic radius is one-half the radius of the inscribed circle, The bottom width of this ideal section, as shown in Fig. 74, is $1.15R$, the top width is $2.30R$, and the side slopes are approximately 0.58 on 1. Although it is rarely possible to use this form of cross-section, its hydraulic value should be recognized. For side slopes of 1 on 1, the most efficient and economical section is that whose base is 0.828 times the depth. The top width of this section is equal to the sum of the two side slopes, and the hydraulic radius is equal, approximately, to one-half the depth.

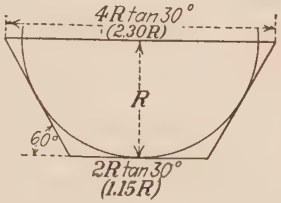


FIG. 74.—Trapezoidal section of highest hydraulic efficiency.

Side Slopes.—After the slope and depth of ditch have been determined, the next element to consider is the slope of the sides. This is governed primarily by the stability of the soil, and secondarily by the method of construction. The side slopes of a ditch should be such that the banks will neither cave nor slide when the material composing them is saturated with water, a condition which exists in the winter and spring and after flood flows have subsided. For the same side slope, the banks of deep ditches are more likely to cave than those of shallow ones, and to this extent the depth and slope of the sides of the ditch are interrelated.

For ditches constructed through muck and peat soils, side slopes of ¼ on 1 have been used satisfactorily. Steep side slopes are desirable in this case, since they prevent the growth of weeds which is the most serious source of ditch deterioration in such soils.

In stiff clay, the banks will stand at a slope of ½ on 1 if the ditch is not too deep. This slope is about that which is ordinarily constructed by a dipper dredge, and approximates closely the slope of maximum efficiency.

A slope of 1 on 1 is more widely used than any other in ditch design, even though, as actually constructed, the banks are steeper and the bottom wider than specified in the plans. Most earth will stand at this slope. The bottom width of the ditch has a bearing on the side slopes, since the caving of the banks is more serious in narrow ditches than in wide ones. A slope of 1½ on 1 should be used for small ditches through soils where a slope of 1 on 1 would be satisfactory for large ones.

If the material is sand, sandy loam, or porous clay, slopes of 2 on 1, or even 3 on 1, will be necessary. Slopes of 3 on 1 were found necessary in three drainage districts in Pemiscot County,[1] Missouri, where the soil encountered was principally a joint or "buckshot" clay. This clay is composed of very fine particles and is very porous, having numerous joints or cracks through which water enters readily, causing the clay to become soft and mushy. Where slopes greater than 1 on 1 are used, it is advisable to use a dragline excavator in their construction.

Thus, it is seen that the slope of the banks is a function of the stability of the soil and should not be determined arbitrarily and without proper investigation. Steep side slopes have been

[1] *Eng. News-Record*, p. 227, Aug. 11, 1921.

the source of failure of many ditch systems and the direct cause of high maintenance costs. Because of this, the present tendency is to specify slopes of 1½ on 1 and 2 on 1, where the stability of the material is doubtful.

Much of the difficulty which has been experienced in maintaining ditches has been due to the method of construction. Until recently, the floating dipper dredge was used almost exclusively in ditch construction, and since it is difficult to construct slopes of 1½ on 1 with this machine, and impossible to construct flatter slopes, contractors have preferred to excavate a channel with greater bottom width and steeper side slopes than those called for in the plans; and engineers and drainage commissioners have permitted them to do this under the impression that the larger cross-section more than offset the objections of caving banks. The theory was that the banks would slide until a stable condition was reached, that the sliding material would come to rest at the foot of the slope, and that eventually the standard cross-section would be approximated. This theory, however, does not work out in practice. The material from the caving banks frequently becomes so soft that when it slides into the water it flows over the bottom of the ditch to, or beyond, the center, thus raising the bottom of the ditch, decreasing the slope and the velocity of flow, and greatly impairing the efficiency of the ditch. As a result of experience, engineers are now requiring the side slopes to be constructed more carefully so as to reduce caving and sliding to a minimum.

Bottom Width.—The bottom width of a ditch is dependent mainly upon the discharge. For the tentative slope and depth and fixed side slopes, a bottom width is computed such that the ditch will carry the estimated discharge at about 0.8 or 0.9 of the bank-ful stage, thus providing a factor of safety. If the bottom width thus determined does not give a satisfactory cross-section, the depth and slope are modified, if it is possible, so as to give a more efficient section.

The bottom width necessary for a given discharge, slope, depth, side slope, and roughness factor can be computed from one of the formulas for flow given on pages 110 to 135, or more conveniently, can be obtained from a diagram such as that given in Fig. 75. This diagram which was designed by F. T. Mewes[1] is based on Manning's formula and is a combination of a logarith-

[1] *Eng. News-Record*, p. 173, Jan. 25, 1923.

mic diagram and a nomographic diagram, having a common hydraulic radius scale. The diagram as given is plotted for side slopes of 1 on 1, which is the most commonly used slope. For other side slopes, that portion of the diagram plotted on logarithmic paper will be changed, but the nomographic diagram to the left—including the hydraulic radius scale—will remain unchanged.

In using the diagram, a bottom width is first assumed, for example 16 feet as shown at *A*. A vertical line is drawn through

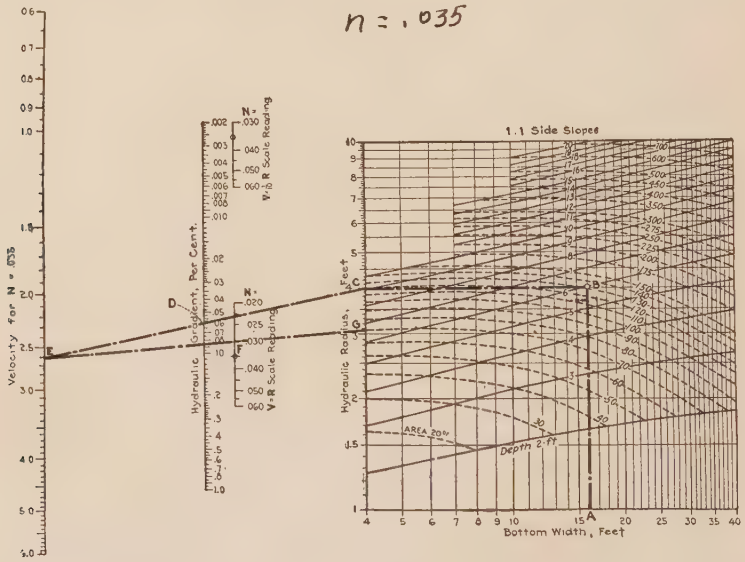


FIG. 75.—Diagram for designing open ditches. (Based on Manning's formula.)

A until it intersects the inclined line representing the given depth of flow, as the 6-foot depth line at *B*. Since the dotted curved lines represent area of cross-section, the point *B* gives the area of cross-section as 132 square feet. A horizontal line is projected from *B* to the hydraulic radius scale at *C*, and the hydraulic radius is found to be 4 feet. From *C*, a line is drawn through *D* on the gradient scale to the velocity scale at *E*. The given grade in the example is 0.06 per cent, and the scaled velocity is 2.61 feet per second. The velocity, 2.61, multiplied by the area, 132, gives the discharge as 344 second-feet. The velocity scale is designed for a roughness factor, *n*, equal to 0.035. For other values, for example *n* equal to 0.030, a line

is drawn from E through F on the n scale to G on the hydraulic radius scale, and the velocity obtained from that scale.

Although hydraulic efficiency is the principal object sought in ditch design, economy of construction is likewise an important factor; and frequently lower costs can be obtained by small changes in design which will not appreciably affect the efficiency. For example, each type and size of excavating machine has definite limits of operation as regards maximum and minimum width of ditch which it can dig, and the maximum distance which it can dig below the water surface and raise the dredged material above the surface; and by changing the width or depth a small amount, a smaller machine or one of a more available type can do the work, and at a lower cost than would have been possible in the original design. The limitations of excavating machines are given in Chapter XIV on Excavating Machinery.

The minimum bottom width should be about 4 feet, although many ditches have been constructed with bottom widths of 2 and 3 feet, generally with unsatisfactory results. Where a smaller cross-section than that given by a bottom width of 4 feet and a depth of 6 feet is required, a large tile drain should be used instead of an open ditch.

Design Factors.—The conditions which must be fulfilled by the ideal ditch are:

1. The bottom of the ditch must be low enough to drain the land which it serves.

2. The ditch must have sufficient capacity at all points to carry the water brought to it within its banks.

3. The side slopes must be such that the banks will neither cave nor slide.

4. The velocity of flow must be such that neither serious scouring nor silting will occur. Scouring is not objectionable if it takes place gradually and if the scoured material is carried in suspension beyond the end of the system; or unless extensive scouring of banks results in the caving of the spoil banks. On the other hand, silting, beyond that required to smooth over the minor irregularities in the bed and banks made during construction, is extremely undesirable, since the efficiency of the system is thereby impaired.

It occasionally happens that it is impracticable, if not impossible, to drain all the area. For example, there may be a pond in the area whose bottom is considerably lower than the surround-

ing land, and to construct the ditch low enough to drain the pond completely would increase the cost of the ditch by an amount greater than the value of the additional land reclaimed. In this event the pond would be only partially drained.

Since the inflow to a ditch increases from the upper to the lower end, the cross-section of the ditch should also increase downstream. Naturally, such increases in discharge are most marked at the outlets of tributary ditches. All necessary enlargements of channel should be made gradually so as to avoid the loss of head caused by the eddies which form at a sudden enlargement of section.

The proper side slopes can be determined by proper investigation of the nature and stability of the soil.

As to maintenance and efficiency of operation, the velocity factor is the most important, since upon it depends the stability of the channel section. This topic is of sufficient importance to warrant discussion in a separate paragraph.

Velocity.—The ideal or critical velocity of flow is never attained, since scouring and silting action is so complex and varies so with different materials and at different stages of stream that it is impossible to state with any degree of certainty just what the best velocity is for any given case. And even were it known, it could not always be used because the more or less fixed elements of slope, depth, and discharge might preclude its use. In any problem, nevertheless, the most probable value of the critical velocity should be chosen and the other factors of design modified as far as possible to agree with it.

In the author's opinion, Kennedy's formula for the critical velocity is the most reliable information on this subject. The investigations on which Kennedy's conclusions are based are discussed on (pages 141 to 144,) and the formula is repeated here for convenience.

$$V_o = 0.84d^{0.64} \tag{33}$$

The formula shows that scouring and silting depend upon the depth of flow as well as upon the velocity; and it follows that in the design of ditches a change in velocity must be accompanied by a change in depth, if silting and scouring are to be prevented.

(In practice, the critical velocity for the maximum depth of flow is obtained from the formula or from the curve in Fig. 69, and a bottom width is chosen such that the given discharge will

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be carried at as near that velocity as the other factors will permit. As shown in Fig. 69, when the slope is less than about 0.0003, it is impossible to obtain the critical velocity with a practical cross-section, unless the discharge is quite large, necessitating an unusually wide channel. Hence, for ditches with flat slopes, some silting cannot be prevented. When the slope is 0.0004 or greater, it is possible to secure a practical section which will give the critical velocity if the discharge is large. For small ditches, however, the critical velocity can be obtained only by constructing very shallow ditches which are difficult to maintain and which provide inadequate outlets for tile drains. Velocities ranging from 2 to 4 feet per second are used satisfactorily in alluvial soils. Kent[1] gives the following table of safe velocities for alluvial soils.

TABLE XXX.—SAFE VELOCITIES OF FLOW IN ALLUVIAL SOILS

Material	Safe velocities in feet per second
Pure sand	1.1
Sandy soil, 15 per cent clay	1.2
Sandy loam, 40 per cent clay	1.8
Loamy soil, 65 per cent clay	3.0
Clay loam, 85 per cent clay	4.8
Agricultural clay, 95 per cent clay	6.2
Clay	7.3

When there is a change in the slope but no change in the depth of flow, the width must be changed to keep the velocity unchanged, so that the velocity-depth relation will remain the same, or else silting or scouring will occur. Also, when the depth is increased, the velocity must also be increased by narrowing the channel.

To operate efficiently, a ditch system must be self-cleaning. The depths and velocities throughout the system must be so adjusted that any material washed into any part of it will be carried to the main outlet and beyond. The chief difficulty is experienced at the junction of channels, and the velocities at these points should be given careful consideration. Although

[1] KENT, "Mechanical Engineers' Pocket Book," p. 755.

it is impossible to design a channel such that no deterioration will occur, an intelligent use of the above principles will largely eliminate the most serious difficulties.

Illustrative Example.—It is required to design a ditch in northeastern Arkansas to drain a flat area of 30 square miles. The tentative profile gives a gradient of 0.0003 and a minimum depth of channel of 9 feet. Since it is not desirable to have the water at a bankful stage, a depth of 8 feet will be used, leaving the additional foot as a margin of safety. The soil is such that side slopes of 1 on 1 can be used safely. A value of n equal to 0.030 is assumed as being a safe value for ditches in this soil.

Since curve 5 in Fig. 51 was derived from run-off data obtained in this locality, it may be used to determine the run-off. From this curve, a run-off of 12.4 cubic feet per second per square mile is found to apply to a drainage area of 30 square miles. This coefficient gives a discharge of 372 cubic feet per second to be carried by the ditch.

From the diagram in Fig. 75, it is found by trial that a bottom width of 12 feet gives an area of 160 square feet, a hydraulic radius of 4.65 feet, and a velocity of 2.35 feet per second. This velocity and area give a discharge of 375 cubic feet per second, which agrees with the required discharge; hence a bottom width of 12 feet meets the conditions. The critical velocity curve in Fig. 69, however, shows that for a depth of 8 feet, a velocity of 3.2 feet per second is required to prevent silting. At this velocity, a discharge of 372 cubic feet per second would be carried by a channel 116 square feet in cross-section; and for a depth of 8 feet this would mean a bottom width of 6.5 feet and a slope of about 0.00065. This section is practically that of maximum efficiency since the bottom width is nearly 0.8282 times the depth (see page 175). Although the given slope of 0.0003 is tentative and possibly can be increased somewhat, it is evident that it cannot be increased to 0.00065, and, therefore, if the critical velocity is to be attained more closely the depth of channel will have to be decreased. If the slope of 0.0003 remains unchanged, a depth of 4 feet and a width of 44 feet will give a discharge of 375 cubic feet per second, and a velocity of 1.95 feet per second. For a depth of 4 feet the critical velocity is 2.04 feet per second. It is just as impossible to reduce the depth to 4 feet as it is to increase the slope to 0.00065, therefore in this problem the critical velocity cannot be obtained and some silting will occur. The

tentative slope of 0.0003 should be increased and the tentative depth of 8 feet decreased as much as practicable, and the problem solved with the new values. Assuming that the tentative slope and depth could be changed to 0.0004 and 7 feet, respectively, a channel of 13-foot bottom width would carry the required discharge at a velocity of 2.64 feet per second, which agrees more closely with the critical velocity, which is 2.92 feet per second for a 7-foot depth.

Ditch Junctions.—One ditch should not enter another at right angles, because of the retardation of flow in both channels caused by the impinging of each current upon the other. The current of the main channel would be forced over toward the opposite bank and the stability of that bank endangered. The decrease in velocity at this point would result in silting and the consequent impairment of both channels.

The two channels should join at an angle of about 30 degrees, and this can be effected by introducing a curve in the alignment of the smaller channel near its end.

The bottoms of the two ditches at their junction should be at the same elevation. This can be accomplished, even in the case of shallow laterals, by increasing the grade of the lateral in the last 200 or 300 feet. While such a change in the grade is not desirable, it is better than the use of an overfall or drop at the junction, especially if the discharge of the lateral is large. The general result of a drop is the formation of a bar of silt just below the junction, unless the entrance is protected by a timber or concrete wall and apron to prevent scouring.

Berms.—The material taken from a ditch is piled on each side and forms artificial banks, which, if continuous, give added protection to the land when the ditch overflows its banks. To prevent this material from washing and sliding back into the ditch, it should be deposited so that there will be a clear space, or berm, at least 10 feet in width, between the foot of the inner slope of the waste bank and the edge of the bank of the ditch. Where this is done, little of the waste material will wash into the ditch, unless excessive caving of the banks occurs. The steeper the side slopes, the greater is the danger of caving, and hence the greater should be the width of the berm.

A berm of 15 feet can be obtained just as readily as one of 10 feet for ditches with bottom widths up to 14 feet, and should be specified.

In swamp soils it frequently happens that the bearing power of the loose saturated soil is not sufficient to sustain the weight of the waste banks, with the result that the latter settle and force up the material in the bottom of the ditch. Where this condition exists, a pilot ditch is first dug, which partially drains the soil and increases its bearing power, and later the ditch is dredged to its full dimensions.

A safe width of berm is an important element in the design of a ditch, as the operation of the system is based on the assumption that the waste material will not fall back into the ditch and clog it. Not only should a safe width of berm be included in the design and stated in the specifications, but also the engineer should see that the contractor adheres to the specifications. Here again the limitations of excavating machines enter the problem. A floating dredge with a boom of given length can deposit material just so far from the center of the dredge and no farther, and if this distance is not sufficient to carry out the specifications, then so much the worse for the specifications, since it cannot be done. The engineer should oppose the letting of a contract to a contractor, even though he be the lowest bidder, if the equipment which the contractor has available for the work is inadequate to carry out the provisions of the specifications in full.

Waste Banks.—As usually constructed, the waste banks are unsightly, rough ridges, paralleling the ditch. It is preferable to spread out and level off the material. This reduces the pressure on the soil near the banks, lessens the danger from caving banks, prevents the waste material from getting into the ditch, and enables the land to be cultivated.

Continuous waste banks prevent the surface water from the adjoining fields from flowing into the ditch. Provision must be made for this water either by leaving openings in the waste banks at the natural depressions and at intervals of about 500 feet where the ground is level, or better, by constructing tile drains under the banks at these points. The latter method prevents the washing away of the waste banks and the bank of the ditch itself, and the deposition of silt in the channel at those points. An open drain only 12 to 18 inches deep will cause trouble. The ends of the tile drains should be properly protected by headwalls at least 3 feet wide and extending a minimum distance of 18 inches below the bottom of the ditch. (The water should discharge on a concrete or timber apron to prevent scouring.)

Ditches in Levee Districts.—When the land to be drained lies in the flood plain of a large stream and is protected from overflow by levees, the water which collects behind the levees is generally removed by pumping. The ditches in this case collect the water from the area and deliver it to the pumps. The outlet of the ditch system is fixed by the location of the pumping plant. Generally this is at the lower end of the district, but occasionally poor foundations or inaccessibility at this point make another location desirable.

The design of the ditches differs somewhat from that of those previously discussed. The slope is small, usually 3 to 6 inches per mile, because a greater slope will deliver the water to the pumps faster than they can remove it. Even level grades have been used satisfactorily, as the lowering of the water in the ditch at the pump intake will produce a head which will cause a flow. Because of the low velocity of flow in such ditches, silting is certain to occur, especially if the silt-laden bluff streams are permitted to discharge into the ditch system. The depth of ditches should be 2 or 3 feet greater than that actually needed to allow for this silting. The ditches are also made wider than the hydraulic requirements demand to provide storage during excessive storms when the rate of run-off exceeds the capacity of the pumps. At the pumping station, a small bay or reservoir is frequently constructed to give additional storage. In this way, smaller pumps can be used and the average head pumped against is less than where such storage is not provided.

Maintaining the Ground-water Level.—The object of a drainage system is to remove from the soil that water only which is injurious to plant growth, and if drainage is carried beyond this point, a direct loss is sustained by the landowner. The injurious water is that which collects on the surface or fills the pore spaces in the upper 3 to 5 feet of the soil for a period long enough to damage the crops. This upper layer of the soil is the area penetrated by the roots of plants and is the source of their food supply. The useful water is that which is held in the root zone by capillary action, and the ground water immediately below. The ground water is the supply from which the capillary water used by the plants is replenished, and if the level of the water table is too low, capillary action cannot supply water to the roots as fast as it is needed, with the result that the plants suffer. It is impor-

tant, therefore, to maintain the ground water level during the growing season.

(Tile drains remove only the free water in the soil above them, but deep, open ditches lower the water table below the tile drains.) This action is especially active when the water in the ditches is low during periods of drought—the very time when the ground water is most needed. The low-water flow is maintained entirely from the ground water supply. That drainage is carried too far in many cases is evident from the fact that during dry seasons crops suffer more from insufficient water than formerly, indicating a general lowering of the water table.

If the stage of the water in the ditches is raised during the period of low flow, the ground-water level can be better maintained. This can be accomplished by constructing a movable dam or water gates in the main ditch of the system. In the winter and spring, the dam can be lowered to permit the maximum flow, and during low-flow periods of summer it can be raised to keep the water at any stage desired. The dam would have to be lowered during summer floods. This plan is followed at present by only a few districts, but its value is apparent and it should be used more extensively.

Diversion Ditches.—A diversion ditch is one constructed to intercept the water which would otherwise flow into a district from the lands outside, and to carry it around the district to the outlet stream below. All districts located in river valleys are troubled more or less with the water which naturally runs into them from the higher land outside their boundaries. In some levee districts it may be possible to operate satisfactorily without pumping, if the hill water can be kept out; or if pumping is necessary, a smaller pumping plant will be required. In such cases, diversion channels should by all means be constructed.

In some instances, the hill water enters the valley through one stream which meanders across the flood plain to the river. If the volume of water is not too great, the main ditch of the district may tap the stream at the bluffs and be made large enough to handle its water as well as that of the land within the district. Where the volume of flood water carried by the hill stream is large, it should ordinarily be carried directly to the river through a floodway inclosed within levees. Frequently, such a stream and floodway will form the boundary line between districts.

If the hill water enters the district through a number of small channels in large enough quantities to be objectionable, a diversion ditch along the foot of the bluff will intercept all of them and carry their waters around the lower (usually) end of the district to the outlet stream below. Whether it will be more economical to divert the hill water than to allow it to enter the ditches of the district and be pumped over the levee is a question requiring careful study.

A diversion ditch is essentially a storm-water channel and its flow is very irregular. The run-off from the hill lands is rapid and the period of concentration is short, so that the water in the ditch rises rapidly, remains at the peak stage for only a short time, and subsides quickly. The run-off modulus will be much larger than for ditches draining level areas. For a specific case, the modulus should not be assumed until the characteristics of the drainage area have been carefully studied and some observations made on the rate of run-off and on the time of concentration of flow during and after a storm, even though it be a moderate one. From these observed data and from the rainfall records of nearby stations, a satisfactory drainage modulus can then be chosen.

The slope and dimensions of a diversion ditch should be such as to give as high a velocity as the soil through which it is constructed will withstand, since the tendency is for silting to occur, because of the large amount of suspended material carried by the swift hill streams.

The material excavated from a diversion ditch is placed on the lower side, leaving as wide a berm as possible, and forms an artificial bank which gives additional protection against overflow.

Sedimentation Basins.—Bottom-land districts are damaged not only by the water from the higher land, but also by the large amount of silt, sand, gravel, and drift carried by the smaller streams from the immediate hills and deposited at the foot of the bluffs where the slope changes abruptly. The ditches connecting these streams to the drainage system soon become filled with sediment unless steps are taken to prevent it. While it is impossible to prevent silt from being carried to the flat valley, it is possible to limit the area of sedimentation by providing a basin or pond, covering several acres of ground, at the foot of the bluffs and at the upper end of the ditch. Such sedimentation basins are surrounded by levees and have a proper inlet for the hill stream and an overflow dam at the entrance to the ditch.

The basin thus formed checks the velocity of the water, the heavier material carried in suspension is deposited, and the water flowing out of the basin into the ditch is free of coarse sediment and drift. The larger the basin, the more complete will be the sedimentation and the clearer the water entering the ditch system. Eventually the basin will become filled with sediment and an adjoining area must be used for a new basin. The deposits in the old basin generally make a very fertile soil which can be cultivated. Diversion channels should also be provided with sedimentation basins where the smaller streams flow into them.

Check Dams.—Erosion and sedimentation can be reduced considerably by the construction of low check dams at intervals in the steep channels which collect the bluff water. These can be constructed of logs, or dry rubble, and should be not over 8 or 10 feet in height. These dams form a series of pools on the hillside, and by retarding the flow of the water reduce its velocity and the scouring action. Such dams are especially necessary at the junctions of these channels with diversion ditches.

Surveys for Construction.—Before a drainage system can be constructed, center line and slope stakes are necessary to indicate to the contractor the location of the ditches and the depths and limits of excavation.

Line Surveys.—The survey starts at the outlet of the main ditch and proceeds upstream. The stationing of each ditch is a unit, that is, the outlet of each ditch is taken as station zero for that ditch. The bearings or azimuths of the tangents can be scaled from the plans, but to avoid accumulative errors it is better to locate the direction of each tangent by north-and-south or east-and-west offsets from definite points or objects on the ground, such as fences, section corners, and transit stations used on the preliminary survey. The tangents are then run to intersections, the intersection angles are measured, the tangent distances are computed, and the ends of the curves located by measurements from the points of intersection. Stakes are placed at intervals of 100 or 200 feet on tangents, and at intervals of 50 or 100 feet on curves, depending upon the degree of curvature. The transit and steel tape should be used in running the lines, and the stakes used should be long enough to be seen readily.

Cross-sectioning.—In describing the procedure of cross-sectioning and setting slope stakes, it will be assumed that bench

marks were established at intervals not exceeding ½ mile during the preliminary survey of the area, and that their accuracy was determined by careful checking.

The grade elevation of every point on the line where center stakes were set can be obtained from the profiles and should be written in column 3 in the cross-section notebook opposite its station, and carefully checked before any cross-sectioning is done. Also the description and elevation of each bench mark should be written in the back of the notebook.

The procedure of cross-sectioning is as follows: Starting with the nearest bench mark, levels are run to the part of the ditch to be cross-sectioned. All level notes incident to carrying the elevations forward are kept in column 2, as shown in the page of sample notes given in Fig. 76. This can be considered a computation column since the backsight and foresight readings are written directly under the elevations of the bench marks and turning points and of the heights of instrument, respectively, and added or subtracted. The only connection between the data in this column and the stationing in column 1 is that each height of instrument is recorded opposite the first station at which it is to be used.

The grade elevation of each station which can be cross-sectioned from that position of the level is subtracted from the height of instrument, and the difference, to the nearest tenth of a foot, is recorded in column 4 opposite the proper station. This difference is called the *grade rod* and is the rod reading which would be obtained if it were possible to hold the bottom of the rod on the bottom of the ditch. Obviously, the difference between the actual rod reading and the grade rod is the cut at that point.

The level rod is held on the ground at the foot of the center stake and read to the nearest tenth of a foot. This reading subtracted from the grade rod at that station is the cut and is marked on the center stake and recorded in column 6 as the numerator of a fraction whose denominator is zero.

The point on each side of the center where the required side slope of the ditch section will intersect the natural surface must next be found and marked by *slope stakes;* and the vertical distances of these points above the bottom of the ditch—loosely referred to as the cut at the slope stake—must be determined so that the volume of excavation can be computed. If the ground

is level transversely of the line, as is ordinarily the case, the cut at the slope stake will be the same as at the center, and the distance from the center stake to the slope stake is found by multiplying the center cut by the side-slope ratio and adding one-half the width of the bottom of the ditch. If the ground is not level transversely, the cut will be different for various points, and the problem is to find that point on each side of the center line whose distance from the center is equal to the sum of one-half the bottom width and the product of the cut at that point and the side-slope ratio. This point can be found only by trial. The cut at the slope stake is recorded in the notes as the numerator of a fraction whose denominator is the distance of the slope stake from the center stake. Generally the three fractions thus obtained constitute the field work at a station. Whenever there is a break, however, in the slope of the ground surface between the center stake and a slope stake, the cut at that point, or points, must be obtained and also its distance from the center. Such data are recorded as fractions and placed between the center fraction and the slope-stake fraction as shown for station 39 + 77 in Fig. 76.

Form of Cross-section Notes.—A very satisfactory form of cross-section notes is shown in Fig. 76. It conforms to the page rulings in the standard field notebook. Column 2 is the only part of the notes which is unusual and needs explanation. All the differential level data are recorded in this column. The first elevation 353.78 is the height of instrument carried forward from the preceding page. Stations 36, 37, and 38 are cross-sectioned from this setting of the level. A turning point is then taken and the foresight reading, 4.46, is written directly under the height of instrument and subtracted from it, giving a value of 349.32 for the elevation of the turning point. The level is moved forward and a backsight taken on the turning point. The backsight reading, 5.12, is added to the elevation of the turning point and gives a new height of instrument of 354.44. This value is recorded opposite station 39, which is the first station to be cross-sectioned from the new setting of the level. This height of instrument can be used up to station 43, when another turn is necessary. The next height of instrument is 355.12 and is recorded opposite station 43.

The five columns devoted to the actual cross-section data give space for recording three fractions on each side of the center, and

where the line crosses a slough or lies in an old channel, this number of readings may be required.

CROSS-SECTION NOTES

Ditch No. 11

Station	Levels	Grade	Grade rod	Left	Center	Right	Areas, square feet	Volumes, cubic yards
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
				8.8	9.2	9.2		
33	353.78	340.56	13.2	—	—	—	228	
	−4.46			16.8	0	17.2		838
37	349.32	340.60	13.2	8.9	9.0	9.1		
	+5.12			—	—	—	225	
				16.9	0	17.1		844
38		340.64	13.1	9.0	9.2	9.3		
				—	—	—	231	
				17.0	0	17.3		820
39	354.44	340.68	13.8	8.4	8.6	8.8		
	−4.18			—	—	—	212	
				16.4	0	16.8		440
+65	350.26	340.71	13.7	3.8 4.5	8.0	8.2		
	+4.86			— —	—	—	153	
				11.8 6.6	0	16.2		58
+77		340.71	13.7	4.0	4.8 4.6	7.8 8.0		
				—	— —	— —	109	
				12.0	0 6.0	12.5 16.0		78
40		340.72	13.7	3.6	3.6	4.0		
				—	—	—	73	
				11.6	0	12.0		268
41		340.76	13.7	3.6	3.6	3.8		
				—	—	—	72	
				11.6	0	11.8		289
42		340.80	13.6	5.2 4.4	4.0	3.8		
				— —	—	—	84	
				13.2 7.0	0	11.8		139
+35		340.81	13.6	8.8 8.4	5.2 4.0	3.8		
				— —	— —	—	131	
				16.8 10.5	0 8.0	11.8		197
+70		340.83	13.6	9.2	8.8 4.4	4.0		
				—	— —	—	173	
				17.2	0 7.5	12.0		224
				9.4	9.2	8.8		
43	355.12	340.84	14.3	—	—	—	230	
				17.4	0	16.8		

FIG. 76.

End Areas.—The area of the cross-section to be excavated at each station is found as follows:

1. *Three-level Sections.*—Where there are just three fractions in the cross-section notes for a station, the following rule is used:

Rule 1.—Area = Sum of the end numerators × ¼ the bottom width + the sum of end denominators × ½ center cut.

2. *Irregular Sections.*—To the fractions given in the notes, the fraction $\frac{0}{b}$ is added to each end, where b is equal to one-half the bottom width. The numerators and denominators of adjacent

fractions are then connected by diagonal lines as shown below. The bottom width is 16 feet and the side slopes are 1 on 1.

$$\frac{0}{8} \quad \frac{8.4}{16.4} \quad \frac{6.8}{7.4} \quad \frac{6.4}{0} \quad \frac{6.6}{5.8} \quad \frac{9.2}{17.2} \quad \frac{0}{8}.$$

Rule 2.—Area = ½(sum of the products of the numbers joined by full lines − the sum of the products of the numbers joined by dotted lines).

The area of the given section is 167 square feet. The accuracy of the field data, that is, obtaining the cuts to the nearest tenth of a foot, does not warrant the computation of end areas closer than to the nearest square foot.

Volumes.—Volumes are obtained by the average end area method. If A_1 and A_2 are the areas of two adjacent sections and l is the distance between them in feet, the volume between them in cubic yards is:

$$V = \frac{l}{27}\left(\frac{A_1 + A_2}{2}\right) \tag{34}$$

For $l = 100$ feet,

$$V = 1.85(A_1 + A_2).$$

If A_1, A_2, A_3, A_4, ... and A_n represent the end areas of a series of sections a *common distance* l apart, the total volume can be obtained more easily from the formula:

$$V = \frac{l}{27}\left(\frac{A_1}{2} + A_2 + A_3 + A_4 \cdots\cdots + \frac{A_n}{2}\right) \tag{35}$$

Maintenance of Ditches.—The efficiency of the best designed drainage system will be lowered 30 to 50 per cent in a short time if the system is not properly maintained. Periodic maintenance at intervals of several years will not do. Continuous maintenance is the price of a successful drainage system; and in the long run is much less expensive than periodic maintenance.

Maintenance on a ditch system should begin as soon as the construction work is completed. It should be seen to that no logs, stumps, lumber, or other débris are left on the banks, berms, or waste banks to be washed into the ditch at the first high water. Also, all temporary dams and foot bridges used during construction should be removed. The commissioners should not allow the individual landowners to place any obstructions in, or across, the channel, such as fences and improperly constructed crossings, or to discharge their tile drains into the ditch except through a properly constructed headwall which will prevent washing of

the banks and scouring of the bottom of the ditch. After each high water, the system should be patrolled, and any trees, logs, brush, or other débris washed in and caught in the channels should be removed. If these precautions are taken, silting will occur slowly in a correctly designed system, and the ditches will require re-dredging only after a long period.

Silting is always most pronounced just below the entrance of tributary ditches and tile drains. The bars formed at these places should be removed annually.

If the slopes of the ditches are low and the depths shallow, difficulty will be experienced with aquatic growths in the channel. Willows, shrubs, and weeds also obtain a foothold on the banks and in time seriously affect the discharge of the channels. Usually, the banks will need to be cleared every 3 to 5 years, and the weeds in the channel cut every year. To prevent willow stumps from sprouting, poisoning the split stump with a solution of granulated caustic soda and white arsenic has been resorted to in several Mississippi districts.[1]

Ditches constructed on flat grades and in light, easily eroded soils will sooner or later become so filled with silt, and their efficiency so lowered, that they will have to be cleaned out. Where the ditches are short and the aggregate amount of excavation is small, it is possible to remove the silt during the dry season with teams and scrapers. Various types of drags or scrapers have been devised for stirring up the silt in the bottom of small ditches so that it will be carried away in suspension by the water; but the application of such devices is limited. The customary method is to re-dredge the channel. Because of the relatively small amount of material removed, the unit cost of excavation will be several times that of the original work.

[1] *Eng. News-Record*, vol. 87, No. 8, p. 322.

CHAPTER VIII

SOIL PHYSICS

The primary purpose of drainage is to carry off the water in the soil which is injurious to plant growth. This is accomplished by laying tile drains at such depths and intervals as to prevent the water table from rising within a certain distance of the surface. The determination of the proper depth and spacing of tile drains is the most important and the most difficult problem met with in the design of tile-drainage systems. The chief factor in deciding this question is the structure or physical composition of the soil; hence it is necessary for the engineer to understand the subject of soil physics before he can intelligently design a drainage system. In attempting to cover in a single chapter such a large subject as soil physics, it is evident that only the more important phases can be discussed, and those only briefly, leaving the more extensive data on the subject to the standard treatises on soil physics, which should be in the drainage engineer's library.

Soil.—The term soil, as applied to agriculture, is that part of the earth's crust which is penetrated by the roots of plants, and from which they receive nourishment. Soil is derived from rocks which have been disintegrated by physical agencies and decomposed by chemical agencies, and mixed with more or less organic matter. Its composition varies with the kind of rock from which it was derived and with the agencies of disintegration, decomposition, and transportation. The large majority of soils are composed of disintegrated materials which have been carried by wind, water, and ice hundreds of miles from the places of formation. Swamp and marsh soils are examples of soils which have been formed in place. In these undrained areas, organic matter derived from mosses, grasses, and other vegetal growths has accumulated in large quantities and forms a large portion of the soil.

The soil in humid regions is not uniform in physical composition and structure from the surface downward, due to the action of plants and animals and to the effect of percolating water

which carries down with it the finer soil particles near the surface. On this basis of distinction, the soil consists of two layers: (1) the *topsoil*, which extends to a depth of about 20 inches; and (2) the *subsoil*. That portion of the topsoil which is turned by the plow is called the *surface soil*, and the lower portion, the *subsurface soil*. The surface soil is darker in color than the subsurface soil and the subsoil because of the larger amount of organic matter which it contains. The topsoil is the most important stratum from an agricultural standpoint since it contains a larger amount of the elements needed by plants in their growth and since the soil bacteria are more numerous and more active in this layer. Where surface erosion has occurred the more fertile surface soil may have been removed, leaving the subsurface soil and sometimes the subsoil exposed. The line of demarkation between the two strata is indicated by a change in the color and structure of the soil. In swamp and bottomland soils, the topsoil is of much greater depth than in other soils.

In acid soils, the subsoil is likely to be very dense and almost impervious to air and water. Such soils are difficult to farm successfully and are difficult to drain.

In arid climates, there is little difference in the structure of the soil with depth, since rainfall, which causes the vegetal covering responsible for the organic matter in soils, and which in moving downward through the soil carries the finer particles downward, is very light in these regions. Such soils under irrigation in time become very similar to those of the humid regions.

Classification of Soil Particles.—Soil particles are classified according to size. There is no single classification which is accepted by all soil physicists. Table XXXI gives the classification which has been adopted as standard by the United States Department of Agriculture and that which was worked out by Dr. Cyril G. Hopkins of the Illinois Agricultural Experiment Station. The latter is believed to be the better classification since the ratio between groups is a constant—the square root of 10—while in the former classification, the ratio varies from 2 to 10. The soil whose particles are all between the limits of size for one class is termed a *soil separate*, and must not be confused with the ordinary soil groups defined in the following paragraph.

TABLE XXXI.—CLASSIFICATION OF SOIL PARTICLES

Name	U. S. Department of Agriculture, diameter, millimeters	Hopkins, diameter, millimeters
Fine gravel	2.000–1.000	
Coarse sand	1.000–0.500	1.000 –0.320
Medium sand	0.500–0.250	0.320 –0.100
Fine sand	0.250–0.100	0.100 –0.032
Very fine sand	0.100–0.050	
Coarse silt	.....	0.032 –0.010
Medium silt	.....	0.010 –0.0032
Silt	0.050–0.005	
Fine silt	.....	0.0032–0.0010
Clay	0.005–0.0001	0.0010–0.00001

Classification of Soils.—Soils are composed of varying mixtures of clay, silt, sand, gravel, and organic matter, and are classified according to the amounts of these substances which they contain. The definitions of the more important soil classes are as follows:

Sands contain more than 75 per cent of sand.

Loams contain from 30 to 50 per cent of sand and less than 15 per cent of clay.

Gravelly loams contain from 25 to 50 per cent of gravel, much sand, and little silt.

Sandy loams contain from 50 to 75 per cent of sand and less than 15 per cent of clay.

Silt loams contain 50 per cent or more of silt, less than 15 per cent of clay, and some sand.

Clay loams contain from 15 to 25 per cent of clay, much silt, and some sand.

Clays contain 25 per cent or more of clay and much silt.

Mucks contain from 15 to 35 per cent of organic matter and much clay and silt.

Peaty loams contain from 15 to 35 per cent of organic matter, a large amount of sand, and a little clay or silt.

Peat contains 35 per cent or more of organic matter, and sometimes some sand, clay, and silt.

Soil Types.—While soil classes are based mainly on texture, soil types are based on texture, color, structure, character of subsoil, and sources or derivation. Soil classes are divided into

soil types; for example, clay soils are divided into drab clays and gray clays, and silt loams into black silt loam, brown silt loam, gray silt loam on tight clay, yellow fine sandy silt loam, etc.

Number and Arrangement of Soil Particles.—The size and arrangement of the soil particles and of the voids between them play a very important part in agriculture, since upon them depend the aeration of the soil, the rate of percolation and of capillary action, and the available supply of moisture and plant food.

A soil made up entirely of particles 1 millimeter in diameter would contain approximately 15,600 particles in a cubic inch with a total surface area of ½ square foot; while one made up entirely of particles 0.0001 millimeter in diameter would contain approximately 15,625,000,000,000 particles in a cubic inch with a total surface area of 5,450 square feet. Soils are composed of all sizes of particles, and in an average agricultural soil the total surface area of the particles in a *cubic foot* is about 70,000 square feet.

The particles of an average soil are of all sizes and shapes, and are mixed together in every conceivable manner. In clays and clay loams, which contain mineral colloids, and in silt loams which contain organic colloids, the soil particles are united in groups and clusters by the weak cementing action of the colloids. The colloids are the very finest particles of mineral or organic origin which assume a gelatinous condition when mixed with water. The spaces between the larger groups are filled with smaller ones, and these in turn have still smaller particles between them, and so on.

The surface soil in humid climates is made up of coarser particles than the layers beneath, due to the action of percolating water which carries the smaller particles downward, and is more pervious because of the granulating effect of plowing and of the action of plant and animal life in opening up the soil.

Porosity of Soils.—Between the soil particles are air spaces or pores which are as irregular in size and shape as the particles themselves. The individual pores in coarse-grained soils are larger than in fine-grained soils; but the total pore spaces or porosity per unit volume is greater in the latter soils. Also, the more irregular the shapes of the soil particles, the greater the porosity. The amount of organic matter in a soil affects its porosity, the more the organic matter the greater the porosity. The action of bacteria on the organic matter results in the forma-

tion of colloids, which through their cementing properties cause a granulation of the soil particles.

Mosier and Gustafson[1] give the porosity of different types of soil as shown in Table XXXII.

TABLE XXXII.—POROSITY IN SOILS OF VARIED PHYSICAL COMPOSITION

Soil type	Organic matter content, per cent	Porosity, per cent	
		Loose soil	Compact soil
Sand	0.75	44.9	39.7
Brown sandy loam (medium sand)	2.90	53.7	43.3
Yellow fine sandy loam (loess)	0.80	59.0	49.9
White silt loam	0.79	59.7	50.2
Brown silt loam	4.88	60.4	50.4
Black clay loam	5.50	61.2	52.7
Drab clay	3.60	68.2	58.2
Peat	64.48	65.6	60.8

Soils of medium porosity are better adapted to most crops. Soils with large pore spaces permit too rapid percolation, aeration, and evaporation, and are readily blown by the wind; while those with small pore spaces, such as clays, do not permit sufficient percolation and aeration for the best growth of crops.

Hygroscopic Moisture.—Hygroscopic moisture is that moisture which condenses upon the surface of thoroughly dry soil particles when exposed to the air. The amount of moisture which is thus condensed depends on the temperature and humidity of the air and on the total surface area of the particles exposed to the air. The amount of moisture, expressed as a percentage of the dry weight, which a soil will adsorb from a saturated atmosphere at a given temperature is called the *hygroscopic coefficient.* This moisture forms an extremely thin film over the surfaces of the particles. The larger the percentage of colloids in a soil, the larger the hygroscopic coefficient will be. Hilgard[2] found the

[1] MOSIER and GUSTAFSON, "Soil Physics and Management," p. 184, J. B. Lippincott Company, Philadelphia.

[2] HILGARD, E. W., "Soils," p. 196, The Macmillan Company, New York 1906.

hygroscopic moisture of three soils, which were very much alike except for the amount of iron sulfate which they contained, to be 4.92, 15.40, and 19.66, respectively. The hygroscopic coefficients of several types of soil, according to Briggs and Shantz[1] are given in Table XXXIII, column two.

TABLE XXXIII.—RELATION OF HYGROSCOPIC COEFFICIENT TO THE WILTING COEFFICIENT

Soil type	Hygroscopic coefficient, per cent	Wilting coefficient, per cent	Capillary water remaining, per cent
Coarse sand	0.5	0.9	0.4
Fine sand	1.5	2.6	1.1
Sandy loam	3.5	4.8	1.3
Fine sandy loam	6.5	9.7	3.2
Loam	7.8	10.3	2.5
Clay loam	11.4	16.3	4.9

The thickness of the hygroscopic moisture film is extremely small. For a coefficient of 2 per cent in soils of average specific gravity, the thickness of the moisture film is about 0.000002 inch; and for a coefficient of 10 per cent, 0.00001 inch.

The hygroscopic moisture film is so thin and is held so firmly by adhesion to the soil particles that it cannot be used by the roots of plants; nor is there any movement of water from one particle to another.

Capillary Water.—When a small amount of water is added to a soil which contains the maximum amount of hygroscopic moisture, the thickness of the soil-moisture film is increased; and where particles of soil come in contact, the film is continuous from particle to particle, that is, there is a film around each particle and a "waist" between the particles. Water can now move from one particle to another, though the movement is quite slow. This additional moisture can be used by the roots of plants, but the amount is insufficient to satisfy their needs. The hygroscopic moisture content of a soil must be increased about 50 per cent before there is sufficient moisture in the soil to prevent a plant from wilting and dying. This percentage of

[1] BRIGGS, L. J. and SHANTZ, H. L., *Bull.* 230, Bureau of Plant Industry, United States Department of Agriculture.

moisture is called the *wilting coefficient.* Table XXXIII, column 3, gives this wilting coefficient for several common soil types.

As water is added to a soil with a moisture content equal to the wilting coefficient, the soil-moisture film increases in thickness, the water is held less firmly by the particles, and the capillary movement is easier and more rapid. The capillary movement may take place in any direction, though it acts more readily downward than upward, since it is then working with the force of gravity instead of against it. The capillary pull depends on the curvature of the moisture film; the greater the curvature, that is, the smaller the radius of curvature, the greater the pull. If the curvature of the films on all the particles is the same, the water will be in equilibrium and there will be no movement. The greater the difference between the curvature of film on adjacent particles, the more rapid is the capillary movement. The smaller the amount of capillary water in a soil, the thinner is the soil film, the greater is its curvature, and the stronger the capillary pull. As more and more water is added to a soil above the wilting coefficient, the thicker becomes the moisture film, the larger the radius of curvature, and the less the capillary pull. Presently the point is reached where the capillary pull equals the gravitational pull, and the point of *maximum capillary capacity*, or moisture-holding capacity, is reached. The moisture in the soil at this point is also expressed as a percentage. If additional water is applied, it will slide off the moisture films and be carried downward along the path of least resistance by the action of gravity.

Tests in the laboratory on small samples of soil show that the maximum capillary capacities of soils vary from about 28 per cent for sand to about 67 per cent for black clay loam; but under field conditions, these amounts are much too high. The soil directly above the water table, however, contains the most capillary water, and as the surface is approached the amount of capillary water which the soil can retain becomes less and less, because the capillary pull decreases as the distance above the water table increases. Thus, the maximum capillary capacity of soils varies with the depth. For average loam soils, it is about 20 per cent.

The capillary water is that which is used by plants in their growth. While the amount of water needed varies considerably for different plants, that required for the best growth of the

common crops varies from 40 to 60 per cent of the maximum capillary capacity of the soil. This percentage is called the *optimum water content.*

While the movement of capillary water is more rapid in coarse-grained than in fine-grained soils, the distance through which it takes place is much greater in the latter soils. Very fine-grained soils are an exception, because of the large resistance to capillary movement caused by the extremely small pore spaces. Tests made at the University of Illinois[1] show that for coarse-grained soils, as sands, the upward movement in 1 hour is 7.5 inches; in one day, 9.2 inches; and in 14 days, 10 7 inches. For fine-grained sandy loams, the movements were 11.2, 39.5, and 58.2 inches, respectively, for the same periods of time; and for peat soils, the movements were 2.6, 5.5, and 6.1 inches, respectively.

Gravitational Water.—Soil water above the point of maximum capillary capacity and up to the point of saturation is called gravitational water or free water. This is the water which is

TABLE XXXIV.—MOISTURE CONTENT OF SOILS

Kind of soil	Pore space, per cent	Maximum water capacity, per cent[1]	Maximum capillary capacity, per cent[1]	Maximum gravitational capacity	
				Per cent[1]	Depth in inches in top 4 feet of soil
Dune sand	52	40.5	10.7	29.8	18.3
Coarse sand	51	39.5	10.6	28.9	18.0
Fine sandy loam	50	38.0	18.0	20.0	12.7
Silt loam	50	38.0	20.9	17.1	12.1
Clay	59	54.5	30.4	24.1	7.3
Muck	80	333.0	250.0	83.0	9.6

[1] Percentages are computed by weights.

injurious to plant growth and which is removed by drainage. The percentage of gravitational water also varies with the depth below the surface, the surface soil under saturated conditions containing a larger amount than the subsoil.

[1] MOSIER and GUSTAFSON, "Soil, Physics and Management," p. 207.

Lyon and Fippin[1] give the moisture content of soils as shown in Table XXXIV.

The quantities in the last two columns are the amounts of water in a saturated soil which can be removed by drainage.

Percolation.—The rate at which gravitational water moves downward through the soil depends on the size of the soil particles, the size of the pore spaces, the granulation of the soil, the organic matter in the soil, and the openings in the soil made by cracks, by the burrowing of earthworms, and by the decayed roots of plants. The first-mentioned factor is the most important. In coarse-grained soils, percolation is much more rapid than in fine-grained soils, notwithstanding the fact that the latter soils are more porous.

Both capillary movement and percolation are more rapid when the soil is wet. Thus, directly after a rain on soil which is dry, the downward movement is slow until the soil becomes moistened, after which percolation takes place much more freely.

The rate of percolation is the important factor in the design of drainage systems. Since this varies for different soils, it is necessary to study the soil structure in each case. Soil surveys have been made in some of the states and soil maps have been prepared which show the soil types in colors. These maps are invaluable and should always be used when available. A soil auger should be part of the engineer's field equipment, and samples of the subsurface and subsoil should be taken at representative places and the texture of the soil studied. If possible, laboratory test should be made on the percolation of water through the soil samples and the results compared with those obtained from some soil taken as standard; or better still, with samples taken from a field which is satisfactorily drained by a system of known depth and spacing of tile. In this way, in the course of time, the engineer can obtain a large amount of valuable data on the percolation of water through the various types of soil.

Plant Food.—All plants are dependent for their growth on ten elements. These are carbon, oxygen, hydrogen, nitrogen, phosphorus, potassium, magnesium, calcium, sulfur, and iron. The first three are obtained from air and water, and the others from the soil. Some plants (alfalfa, clover, peas, and other legumes) can obtain nitrogen from the air, if the amount liberated

[1] Lyon, T. L. and Fippin, E. O., "Principles of Soil Management," p. 162; The Macmillan Company, New York, 1909.

by the soil is insufficient. This is accomplished by nitrogen-gathering bacteria which live in the nodules on the roots.

These elements cannot be used for plant food in their original form, but must first be changed into soluble phosphates, nitrates, and salts of potassium, magnesium, and calcium. This change is brought about by the bacteria which live in the soil and decompose the organic matter into organic acids, such as carbonic and nitric acids, which can dissolve the elements in the soil and thus furnish plant food. The more rapid the liberation of plant food, the more rapid is the growth of plants. For a given soil, the rate of liberation depends on the temperature of the soil and on the amount of water and air which it contains. Since the plant food is liberated through the life activities of the nitrifying bacteria, and since the bacteria live on the organic matter in the soil, the more organic matter there is in the soil the more rapid the rate of liberation. The organic matter supplies the elements carbon and nitrogen.

How Plants Grow.—After the elements of plant food have been changed into soluble salts through bacterial action, they are dissolved by the capillary water and held in solution in the films of moisture surrounding the soil particles. The solution is absorbed by the plant only through the root hairs, which are clusters of fine hairs growing at the tips of the young roots. These roots are immersed in the moisture films and are active only while growing, and lose their power to absorb water when they get older. If for any reason the root ceases to grow, no water is taken into the plant and it also ceases to grow. The food-laden water is taken into the plant by osmosis, which is a form of molecular attraction. If two liquids of different concentrations are separated by a membrane, there is a movement of liquid through the membrane from the thinner to the thicker fluid. The concentration of the solution within the root hairs is higher than that without, hence there is a movement of the food-laden soil water into the plant. The solution moves upward at the rate of several feet an hour. The water is transpired through the pores in the leaves, and the food remains in the plant and increases its growth. The amount of water required to mature a plant is astonishing. From 300 to 500 pounds of water are required to produce 1 pound of dry matter.

The greater the difference in the strength of the solution within the root hairs and that without, the more rapidly will water be

taken into a plant. In fertile soils, the solution of plant food is more concentrated than is the case in less fertile soils; hence the former soil requires a smaller amount of water to raise a crop than the latter, which is an added inducement to the farmer to keep his land in as fertile a condition as possible.

If the soil solution is stronger than the root solution, water will be drawn from the plant. This is the action which takes place in strongly alkaline soils, and results in the death of plants.

Results of Drainage.—Bacteria cannot carry on their activities in saturated soil, since the water excludes the air; and soils which are water-logged will not produce the ordinary crops. The results of drainage are: (1) the removal of gravitational water; (2) an increase in the amount of available capillary water; (3) an increase in the volume of soil from which the roots of plants can obtain food; (4) an increase in the temperature of the soil; and (5) the aeration of the soil.

Removal of Gravitational Water.—It should be clearly understood that drainage simply removes the free water in the soil. that is, the water in excess of the maximum capillary capacity, It cannot remove the capillary water, and there is no danger of draining a soil too dry by placing the drains close together. The depth of drains, however, determines the height of the water table and also the amount of water which can be held by capillarity. For example, when the water table is 3 feet below the surface there is more capillary water in the top foot of soil than is the case when the water table stands at 4 feet below the surface; but the total volume of capillary water is greater in the latter instance. It has been found by experiment that the common crops grow best when the percentage of water is equal to the optimum water content, which is about 50 per cent of the maximum capillary capacity a short distance above the plane of saturation.

Thus for each soil, there is a depth of water table which is best for plant growth in that soil, and it is for the engineer to determine that depth as nearly as may be by a study of the soil structure, by tests, and by drawing on his own experience and that of others.

Increase in Capillary Water.—The removal of the gravitional water by drainage increases the amount of available capillary moisture in the soil instead of decreasing it. When the water table is within 1 foot of the surface, the roots of plants will be

confined to the top foot of soil and the amount of available water will be the capillary water held in the top foot of soil. When the water table is 4 feet below the surface, the capillary water in the 4 feet of soil is available for the use of plants; and while the amout of such water per foot of depth may be less, the total amount of capillary water in the 4 feet is much greater than that in the top foot in the former case. Thus it is that drainage increases the amount of useful soil water.

Increase in Available Plant-food Supply.—The roots of plants will not penetrate into a permanently saturated soil stratum, because of the absence of air. Plants must have air to live on, and if deprived of air for a period of 24 to 48 hours they will die. Drainage, by lowering the plane of saturation, increases the volume of soil from which plants can obtain food. Not only is the volume of food increased, but also the rate at which the food is liberated is increased, due to the higher temperature of a drained soil and to the larger supply of air.

Soil Temperature.—The temperature of soils plays an important part in the germination of seed and in the growth of plants.

The surface soil is heated by the absorption of solar radiation. By conduction this heat is slowly imparted to the soil beneath. A dry soil will absorb more heat than a wet one. About four times as much heat is required to raise the temperature of water 1°F. as is required to raise the temperature of dry soil 1°F.; and about twice as much is required to raise the temperature of soil containing 20 per cent of moisture as is required for dry soil. Thus a soil which is saturated warms up very slowly, because the solar radiation is used largely in evaporating the water. Such soils are called cold soils. Drainage by removing the excess water enables a soil to warm up more rapidly. The difference in the temperature of drained and undrained soils at various depths is shown in Table XXXV.

The minimum temperature at which seed will germinate varies from about 41°F. for wheat to about 48°F. for corn. A drained soil will attain these temperatures earlier in the spring than an undrained one, and germination will take place more rapidly in the former soil. Because of the slow germination in a wet soil, the seed are more likely to decay. Experiment shows that wheat requires about 6 days to sprout at a temperature of 41°F., 3 days at 51°F , and 2 days at 60°F.; while corn requires 11¼

days at 51°F., and 3¼ days at 60°F. Thus the effect of the temperature of the soil on the sprouting of plants is seen to be of considerable importance.

TABLE XXXV.—EFFECT OF DRAINAGE ON TEMPERATURE OF SOILS[1]

Time	Thermometer 1 inch below surface		Thermometer 2 inches below surface		Thermometer 4 inches below surface	
	Drained	Undrained	Drained	Undrained	Drained	Undrained
6 a.m.	46.7	45.0	49.0	47.2	52.5	50.0
8 a.m.	58.0	52.5	53.0	50.5	51.0	49.5
10 a.m.	75.0	67.0	66.5	61.0	57.5	54.0
Noon	85.5	73.5	77.0	68.5	67.0	62.0
1 p.m.	87.5	73.8	79.4	70.7	70.6	65.1
2 p.m.	84.0	72.0	81.0	72.0	74.2	68.0
3 p.m.	81.0	70.0	80.0	71.0	76.5	70.5
4 p.m.	76.1	65.1	77.0	68.5	76.8	70.8
5 p.m.	71.2	63.8	74.2	66.3	75.4	70.0
6 p.m.	68.0	62.0	72.1	65.0	74.0	69.0

[1] MOSIER and GUSTAFSON, "Soil Physics and Management," p. 224.

The soil bacteria are more active at higher temperatures and supply a larger amount of plant food. They are most active at temperatures between 80°F. and 95°F.

Aeration of Soil.—In average well-drained soils, the pore spaces amount to about 50 per cent, of which 25 per cent is filled with capillary water and the remaining 25 per cent with air. The roots of plants and the bacteria which prepare the plant food must have oxygen. This is one of the most important elements of plant food and is taken into the plant through the root hairs. As oxygen is taken in, carbon dioxide is given off, which unites with the soil waters to form carbonic acid which dissolves the mineral elements of the soil. The air also contains the nitrogen which is needed in the process of nitrification, or the formation of nitrates in the soil. If too much carbon is liberated and the diffusion is slow, the percentage of oxygen in the soil air is decreased and the plants suffer; hence a soil must be aerated or

ventilated for the best results. Drainage accomplishes this. After each rain, the water in percolating through the soil dissolves the carbon dioxide and carries it away through the tile. As the water passes downward in the soil, fresh air is drawn in. The changes in the pressure of the atmosphere from day to day also assists in ventilating the soil.

Water Table.—The water table is the surface formed by the water in a saturated soil. Its height can be determined at any point by digging a hole and noting the height to which the water will rise in it. The water table is not a level surface but an undulating one, the peaks occurring normally at the ridges or watershed lines, and the depressions at the channel of streams or ditches. If the land is tile drained, the depressions will be at the tile lines and the peaks midway between such lines. In an undrained soil, the water table may be quite near the surface and, directly following a heavy rain, at the surface itself or even above the surface at low spots, where the water collects in shallow pools. Through evaporation and transpiration this water table will be gradually lowered until the next rain again raises it.

In tile-drained soil, the permanent water table forms a slightly undulating surface with the depressions at the elevation of the bottom of the drains, or slightly below during extended dry periods. The slope of the undulations will vary with the structure of the soil. After a heavy rain the water table will rise and will become more undulating, but only for a short period, since the excess water moves toward the tile drains and is carried away by them (see Fig. 77). The soil directly over a tile drain is always in better condition than that midway between drains, since the excess water is carried away more quickly; and in many fields the location of drains is plainly indicated by the more rapid growth of plants along those lines. This subject is treated more fully in the chapter on Underdrainage.

Relation of Soil Structure to Drainage.—From the foregoing discussion it is evident that the relation between soil structure and methods of drainage is a very close and important one. In fact, the design of a tile-drainage system is dependent as much on agricultural as it is on engineering knowledge. Unfortunately, there is little information available as to the actual performance of drainage systems. Soil physicists have made laboratory experiments on the movement of water through columns of soils of various types, and their determinations have been very valu-

able in a general way, but laboratory tests have been used only to a minor extent in connection with specific drainage problems. A few investigations of the run-off from tiled areas have been made by the United States Department of Agriculture and the Iowa Engineering Experiment Station, but these are not sufficient to form a basis for definite conclusions.

Sources of Soils

Residual - all sizes of particles.
Alluvial - Any size.
Glacial - Any size.
Wind Blown - fine particles.

CHAPTER IX

UNDERDRAINAGE

Although systems of open ditches as described in Chapter VII, supplemented by small field ditches constructed by individual landowners, have enabled thousands of square miles of land to be farmed which were formerly worthless, the maximum productivity of the soil cannot be secured without underdrainage. This fact must be recognized when the benefits to be derived from drainage are estimated; and the cost of underdrainage must be added to that of open ditches to obtain the total expenditure necessary to bring the land to the point where the maximum benefits are possible.

Action of Tile Drains.—The more or less saturated condition of the soil in wet areas is due to the fact that the surface soil is underlaid with a stratum of soil so impervious that water can pass through it very slowly, if at all, with the result that the plane of saturation is always very close to the surface. Tile underdrains in, or above, this impervious stratum provide artificial channels which collect the gravitational water and carry it away. The water enters the drain through the open joints at the ends of the tile.

Starting with a saturated soil, which is the condition after a protracted period of rainfall, the movement of the water from the surface to the tile is as follows: The water in the soil at the level of the tile drain is under pressure due to the head of water in the soil above. This pressure causes the water in the soil around the tile to enter the joints, since this is the path of least resistance. The relief thus afforded starts a flow of the ground water in the direction of the drain. As the level of the ground water is lowered, however, its surface does not remain horizontal, but becomes undulating, with the depressions over the drains and the summits midway between them. This is because the water in the soil over the drains has a shorter distance to travel to reach them. The surface of the ground water becomes more and more undulating until all the gravitational water in the soil

above the drain has been removed. This position is shown in Fig. 77 by curve 4. From this stage, the ground-water slope

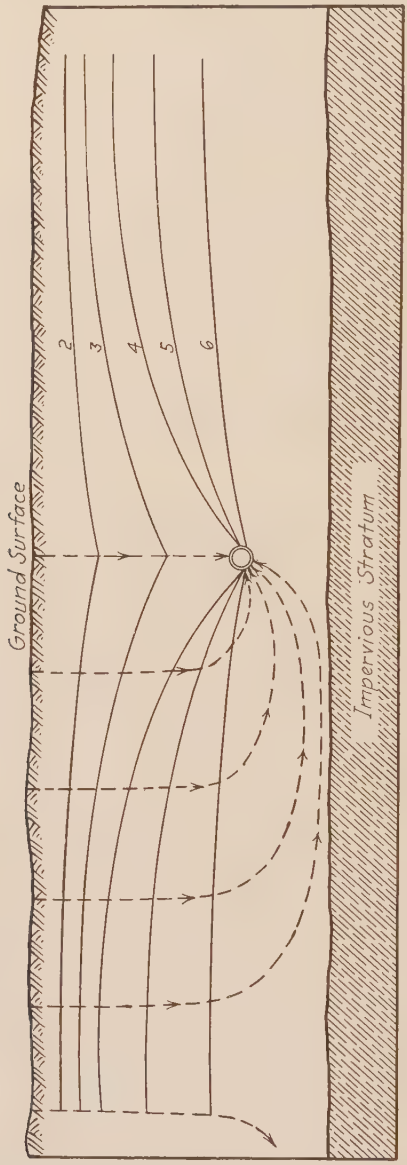


FIG. 77.—Showing successive positions of the water table and the general paths taken by the water in reaching tile.

becomes flatter and flatter as shown by curves 5 and 6 until the pressure tending to cause flow is just balanced by the resistance

to flow offered by the soil particles, and a stage of equilibrium is reached as shown by curve 6. The curvature of the ground-water surface at this stage depends upon the character of the soil; the coarser the soil, the flatter the curvature.

The water in the soil above the drain moves directly downward —as nearly as the irregular passages in the soil will permit—and water enters the drain from the top; but the rest of the water enters the tile from the bottom, the lower water reaching the drain first, because of the pressure of the water above, and the upper water moving diagonally downward and occupying the spaces made vacant by the lateral movement of the water at the level of the drain. The general paths taken by the water, when the water table is maintained at the surface by the precipitation, are illustrated in Fig. 77.

During a rain, the water which percolates into the soil does not reach the water table until the maximum capillary capacity of the soil has been supplied. The topsoil becomes saturated first, causing two planes of saturation—the saturated surface layer and the true ground-water level—with an air-filled soil between them. As the percolation continues, this more or less foul air in the soil is forced out through the tile drains, and the two planes of saturation unite. The water from the surface moves downward through the pore spaces under the action of gravity, and no flow takes place in the drain until this water unites with the ground water, except that caused by water which quickly reaches the ground water through shrinkage cracks, worm bores, and other comparatively large channels in the soil.

Preliminary Survey and Map.—Before a tile-drain system can be intelligently designed, the area must be surveyed and mapped. Since underdrainage generally follows surface drainage, the maps made for the design of the open ditch system can be used, and if such maps are not complete enough for tile-drainage design, they can be supplemented by such additional surveys as are necessary.

The map should show all possible outlets, all property lines, the area of the watershed to be drained by the system—even though part of the area is beyond the limits of the proposed system —and the slopes of the surface by contour lines with a vertical interval of 1 foot. The necessary completeness of the survey and map will depend somewhat upon the size of the area and the expense which is justified for preliminary investigations. Fre-

quently for small areas up to about 100 acres in extent, a sketch showing the elevations of a few controlling points will suffice; but the efficiency of the design should not suffer because of insufficient and inaccurate data. If the landowner is not willing to bear the expense of a proper survey, the engineer should not undertake the work.

Sufficient borings should be made with a soil auger or test pits dug to a depth of 4 or 5 feet to determine the character of the subsoil; and if an impervious stratum is encountered, the elevations of its surface should be marked on the map and subsurface contours drawn. The depth to the water table should also be noted at enough points to determine its surface slope. These investigations will be sufficient in humid regions, where precipitation is the chief source of the water to be removed. In arid regions, however, where seepage from irrigated lands at a higher level and from irrigation canals is the source of the ground water, more detailed information concerning subsurface conditions will be needed, since the location of drains is governed largely by such conditions. If the soil is sandy so that it will not adhere to a soil auger, test pits will be necessary to determine the soil structure and height of water table. The surface of the water table will not conform to that of the ground surface, and ground water contours should be drawn to show its slopes.

Location of Outlet.—Tile-drain systems have their outlets in natural or artificial channels. The latter generally provide better outlets, since they were designed and constructed with that object in view. The first step in the design of a tile system is to chose the best available outlet for the main drain. Where marked natural depressions exist, the location of the outlet is obvious. Where there are no such depressions, the best location of the main drain will determine the position of the outlet. If possible the outlet should be so situated that it will not be submerged.

Tile-drain Systems.—As usually considered, a tile-drain system consists of one or more independent units, each having a separate outlet into an open channel, and composed of a main, submains, and laterals. The area drained by each unit should be as large as the surface conditions will permit, making the number of units for a given area as few as possible.

The laterals constitute the major portion of the system and are without doubt the most important part, as the success of the

system depends upon their efficiency in removing the gravitational water in the soil above them. The determination of the proper depth and spacing of the laterals is the most important and the most difficult problem arising in the design of an underdrainage system. The laterals should be located, as far as possible, along the lines of steepest surface slope, that is, at right angles to the contours, because this arrangement gives a more uniform depth of tile and a steeper grade line. It is impossible, however, to locate all of a series of laterals at right angles to the contours and often angles as small as 45 degrees have been used satisfactorily. The laterals which discharge into the same submain are generally parallel and uniformly spaced.

The chief function of a submain is to receive the water brought to it by the laterals which it serves, and to carry it to the main, which in turn conveys the water from all the submains to the outlet of the unit. On the plans, the mains of the several units are designated usually as *Main A*, *Main B*, etc.; the submains as *Submain No. 1 of Main A*; and the laterals as *No. 1*, *No. 2*, etc., beginning with the one nearest the outlet.

Arrangement of Tile Drains.—The arrangement of the drains is determined largely by the topography of the land and the source of the water to be removed. The arrangements or systems illustrated in Fig. 78—either individually or in combination—have been found by experience to give satisfactory drainage under most natural conditions.

The *natural system* is applicable where the only portions of the area requiring artificial drainage are small, more or less isolated depressions or swamps which receive the natural drainage from the higher lands adjoining. In this case, the main drain follows the largest natural depression from the outlet to the upper end of the area, and smaller drains branch off the main and tap the isolated wet areas. The location of these drains is obvious from the topography, and the lines can be run directly in the field without the use of a map. Where the wet areas are large, the main and submains indicated form the framework of a more elaborate system, including laterals arranged according to one or more of the other systems illustrated.

The *intercepting system* is a special one and is applicable to broad, flat areas which are wet because of the seepage from adjoining high lands. A line of tile properly located at the foot

of the slope will intercept the flow and relieve the wet condition of the low area.

The *gridiron system* is simply an arrangement of laterals entering a submain, and, therefore, applies to only a portion of the area.

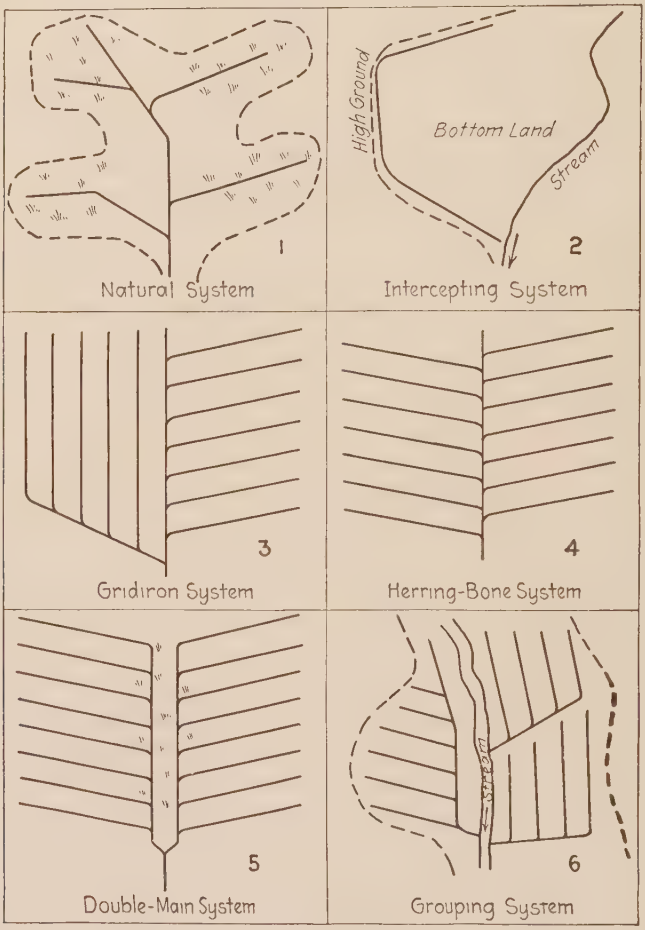


FIG. 78.—Arrangements of tile drains.

It is to be noted that the laterals enter the submains from one side only. This is the most economical arrangement since the land on only one side of the submain is "double drained," that is, a narrow strip of land along the submain is drained by both the submain and the laterals. This arrangement of laterals is used

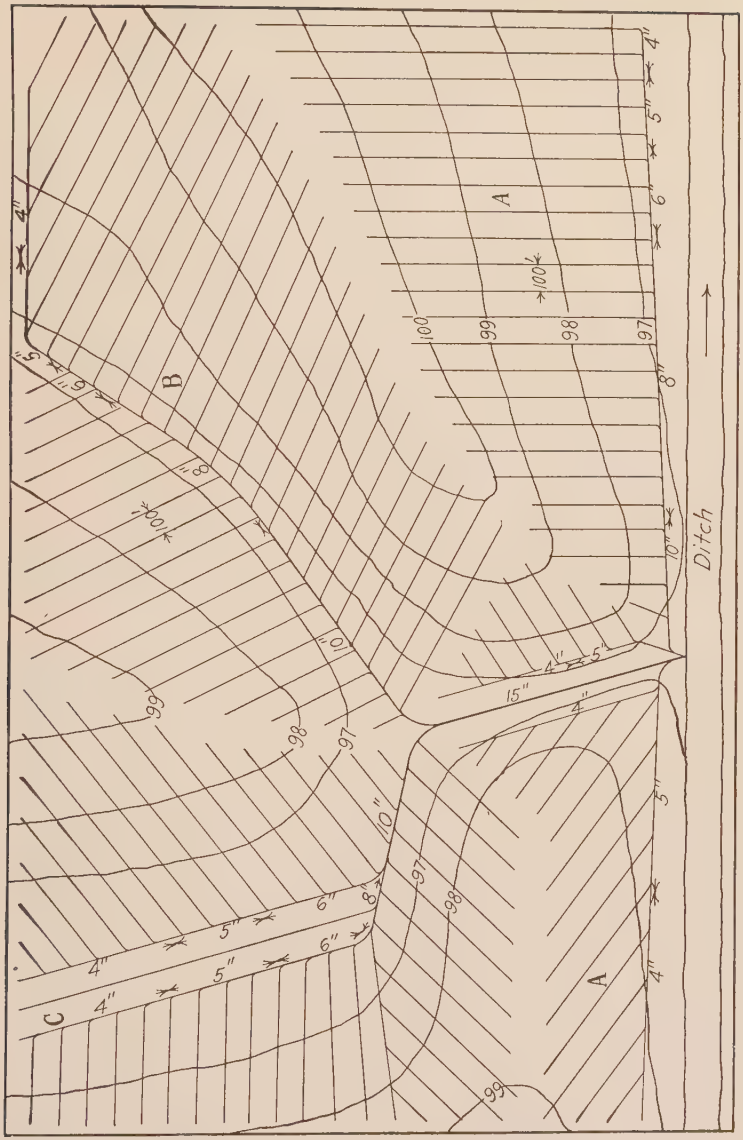


FIG. 79.—Design of drainage system.

either where the surface is level or where the submain is on a gentle slope and the land on one side slopes toward the submain and on the other side slopes away from the submain. If the surface is practically level, the laterals must necessarily be comparatively short (under 800 feet) or otherwise the tile will be too near the surface at the upper end of a lateral and too deep at the lower end. If the surface is such that the tile can be laid at a uniform depth, there is no limit to the length of the lateral. This system is shown at *A* in Fig. 79.

The *herring-bone system* is used where the submain lies in a shallow, narrow depression. In this case the laterals must enter the submain from both sides, as illustrated in Figs. 78 and 79, in spite of the double drainage of the land immediately adjoining the submain. Since, however, the surface water during heavy rains collects in the depression over the submain, this area is wetter and more in need of drainage than the land on the slopes.

The *double-main system* may be considered either a modification of the herring-bone system or a special grouping of the grid-iron system, and is applicable where the depression between two slopes is broad and flat. A single main (or submain) through the center of the flat area would require broken grade lines for the laterals which is undesirable. By laying a submain at the foot of each slope, more uniform grades for the laterals are possible, and smaller tile can be used than would be required for a single main in the center of the flat area.

Depth of Tile Drains in Humid Regions.—Since tile drains are laid in order that the full productivity of the land may be secured, that depth of drain should be used which will hold the ground water at the best level for crop production. If the ground water is too near the surface, the roots of plants will not have sufficient room in which to develop, and their supply of food will be limited; if the ground water is too far below the surface, capillary action will not supply water to the roots in large enough quantities during periods of drought, and the plants will suffer. The roots of different plants penetrate the ground to different depths, and the rate of capillary action varies with the texture of the soil; therefore, the proper depth of drains will depend upon the crops to be raised and the texture of the subsoil. For the best growth of ordinary crops in humid regions, agriculturists agree that the highest portion of the undulating water table should be at least 3 feet below the surface. This would require a depth

of drain of 3½ to 4 feet, depending upon the spacing of the laterals and upon the subsoil structure.

Another factor which must be considered is the time required for the drain to remove the gravitational water from the soil after a heavy rain. This water must be removed before serious injury is done to the growing crops. The rate of movement of the water to the drain is controlled by the openness of the soil, the depth of drain, and the distance between the drains. In open soils, the rate of movement increases with the depth of drain; but for tight soils, the reverse is true. Not only does the depth of laterals determine the level to which the ground water is lowered several days after a storm period, but it also determines the height to which the ground water will rise during the storm. For example, for a given precipitation, the ground water may rise to the surface of the ground if the drain is 3 feet deep, whereas it will be a foot below the surface if the drain is 4 feet deep. Thus, a deep drain increases the zone of soil which is free of gravitational water.

In general, the practice has been to lay drains 3½ to 4 feet deep in loam soils with sandy clay or joint clay subsoils, and 2 to 2½ feet deep in dense clay subsoils close to the surface. The present tendency is to increase the depth of drain in tight soils so that additional plant food may be made available. The movement of the water through the subsoil will be slow at first and the drainage more or less unsatisfactory; but conditions will improve as the soil is gradually opened up by the water and air moving through it—especially if limestone is used on the surface as a fertilizer—and in time the drainage will be satisfactory. The following statement from *Bulletin* 51 of the Iowa Engineering Experiment Station bears out this opinion:

> In most Iowa soils, laterals laid at a depth of about 4 feet will give the best service. In a few cases where the subsoil is very impervious up quite close to the surface, it may be desirable to use less depth, but even in the closest clay it generally will be found advisable to use this depth and decrease the distance between the laterals. It is well known that under-drainage will show far greater results the second and third years than it does during the first year, and it is doubtful if there are any Iowa soils which will not drain satisfactorily after the first 3 years if the system is properly designed and constructed and the laterals laid 4 feet deep.

In draining peat lands, the drains should be laid at a greater depth to allow for the settlement which takes place in these soils

when the water is withdrawn from the large pore spaces, and the vegetable matter of which the soil is largely composed decays. The turf is frequently burned off to a depth of about 1 foot to prepare the land for cultivation. Elliott states that at least 33 per cent of the depth above the drain should be estimated for settling. Capillary action, however, is slower, and operates through a smaller distance in peat soils than in clays and loams, and, therefore, the water table should not be so deep. An original depth of drain of from 4 to 5 feet is the general practice in peat soils, where the thickness of the deposit is from 3 to 5 feet. Tile drains can be laid in or upon the subsoil without construction difficulties and will hold their alignment. Where the deposit of peat is deep, the construction and maintenance of tile drains is difficult, and the efficiency of the drains is uncertain.

The above discussion pertains only to the depths of laterals. The depths of submains is controlled entirely by the depths of the lower ends of the laterals, and they are constructed at a depth which will provide outlets for the laterals. Likewise, the mains are located at depths which will provide outlets for the submains and laterals entering them.

Depth of Tile Drains in Arid Regions.—In general it may be said that drains in arid regions must be deeper than those in humid regions. The depths of 3 and 4 feet used in the East and the Middle West are entirely unsatisfactory in the reclamation of the irrigated lands of the arid West. Wherever irrigation has been practiced for any considerable time, the land is in need of drainage. Not only the irrigated lands need drainage, but also the adjacent lower lying lands which receive the seepage from the higher irrigated areas and from irrigation canals. Frequently, drains are used merely for intercepting seepage water.

The soils in the arid regions are different from those in the humid sections. The arid soils are more uniform in texture and the surface soils are deeper. The dense subsoils found in humid soils at a depth of 2 or 3 feet are either lacking or else occur at a greater depth. Because of the openness of the soil, the plant-root systems extend to greater depths and consequently the water table should be lower. A more important difference in the two soils is that the arid soil contains alkali salts which are injurious to crop growth. Because of insufficient precipitation, these salts have not been washed from the soil as is the case in humid sections. Irrigation, without drainage, has concentrated these

salts at the surface and, as a result, large areas of irrigated lands which produced abundantly in the beginning are now worthless. If the water table is shallow, the soil moisture charged with alkali salts rises too near the surface with harmful results. It is necessary to lower the water table to such a depth that the plant roots will not feed in the alkali solutions. To accomplish this, drains must be laid at a minimum depth of 6 feet. The average depth is generally about 7 feet, and the maximum depth 10 feet.

Numerous borings with a soil auger are necessary to determine the nature and depth of the subsoil. Changes in the soil formation have an important bearing on the depth of drains. For example, suppose borings show a top layer of loam 3 feet thick, underlain by a 3-foot stratum of clay, followed by a 1-foot stratum of sand resting on clay. A drain laid 6 feet deep would be in the top of the sand layer which would provide a poor foundation; and this layer containing seepage water from higher lands would not be drained. In this case, the drain should be laid on the second clay formation at a depth of 7 feet.

The so-called shale lands which are found in all the Rocky Mountain states require special treatment. The surface soil is composed largely of disintegrated shale underlain by shale. The source of water is seepage through the cleavage planes in the shale formation, and drainage is obtained by means of relief wells up to 25 feet in depth, discharging into tile drains at a depth of 6 or 8 feet. For a complete discussion of the treatment of these lands, the student is referred to *Bulletin* 52 of the United States Bureau of Public Roads.

Spacing of Laterals.—*In Humid Sections.*—For a given depth of drain, the spacing of laterals depends upon the amount of water to be removed and its rate of movement through the soil. The former is governed by the amount and intensity of precipitation, the surface slope, and the amount of tributary drainage from adjoining untiled areas—the latter, by the character of the soil, since the texture and structure of the soil determine the resistance to flow and the slope of the undulating surface of the ground water. The finer the soil, the smaller the pore spaces or channels through which the water moves and the higher the resistance to flow. In loams and sands, the channels are comparatively large and the flow is fairly rapid; in dense clays, the pores may be completely filled with capillary moisture and the

flow very slow. Changes in temperature and in barometric pressure also affect the rate of flow, but their effect is small as compared to the other factors involved.

Several formulas have been developed for computing the velocity of flow through soils, but they all require an accurate laboratory analysis, which precludes their general use in drainage work.

For a given soil and depth of drain, the spacing of the laterals will control the slope of the undulating water table, and consequently the rate of movement of the soil water, the rate at which the water table is lowered after a rain, the quantity of water reaching the drain in a given time, and the capacity of the laterals,

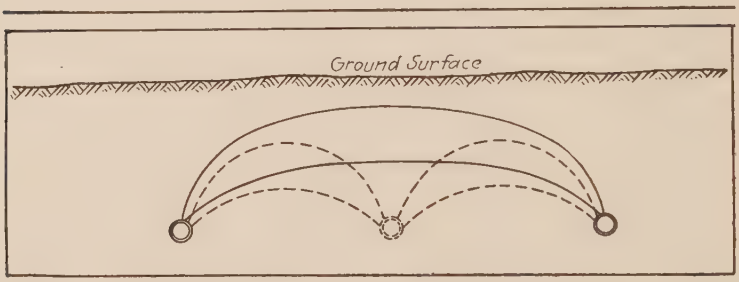


FIG. 80.—Effect of tile spacing on water table.

submains, and mains. The effect of the spacing of laterals on the water table is illustrated in Fig. 80.

It has been found by experience that laterals spaced from 50 to 100 feet apart will give the best results in the large majority of cases. Spacings closer than 50 feet are prohibitive on account of cost, unless the area is to be used for truck crops. If the soil is unusually open, spacings of 150 feet and even 200 feet may be used. The present tendency is to place the laterals closer together. Generally, the closer the laterals, the shallower the depths at which the tile are laid; however, it is doubtful whether this is sound practice in most cases, since the advantages of close spacing are largely lost if shallow depths are used. If the soil conditions are constant and the amount of water to be removed by each lateral is the same, the drains should be uniformly spaced; but, otherwise, the intervals between drains should vary with the conditions. An ideally designed system

should lower the ground water throughout the area at a uniform rate.

Thus it is seen that the texture and structure of the soil are the determining factors in the design of a satisfactory drainage system; hence the engineer should make a careful investigation of the soil. The character of the soil can be determined by auger borings and test pits. The way the auger cuts, the difficulty in pulling it out, and the way the soil sticks to the auger are indications of the soil texture. The soil should be examined by augersful, the auger being withdrawn after two or three turns. Rubbing the soil between the fingers or gritting it between the teeth will determine the presence of sand which is one of the elements controlling the percolation of water. The examination of existing drainage systems in similar soil is possibly the most reliable basis for design.

In Arid Sections.—In irrigated sections, the location of the drains may be more important than the spacing. Frequently, the excess water is due to seepage from higher lands and irrigation canals. If the sand which carries the seepage water is not more than about 8 feet deep, a line of tile laid just below the change in surface slope and at the bottom of the sand stratum will intercept the seepage water and relieve the lower lands of excess water. Where the sand stratum is over 10 feet below the surface, a tile drain is laid at the usual depth and connected to relief wells bored in the bottom of the tile trench to the sand stratum. The water being under pressure will rise in the wells and flow into the tile drain.

To remove the excess water applied by irrigation to the land itself, drains may be spaced from 200 to 500 feet apart. The former spacing applies to clay subsoils, and the latter to sand subsoils. For gravel subsoils, the spacing may be 1,000 or even 1,500 feet.

Run-off Modulus.—The run-off modulus is the depth of water in inches over the underdrained area which must be removed by the drains in 24 hours. A drainage system is designed so that when the main is flowing full this amount of water will be discharged in a day. The run-off modulus is a measure of the maximum rate at which the water will move through the soil to the laterals. The rate of discharge of a tile system is not uniform even for a period of 24 hours, but decreases as the ground water is lowered. The maximum rate of discharge generally

occurs from 5 to 10 hours after the maximum intensity of precipitation, when the water table is the highest. This, of course, depends upon the rate and length of precipitation.

Too often, engineers determine run-off from a study of the precipitation only; but from the foregoing discussion it is evident that the depth and spacing of drains are factors which must also be considered, especially the latter. Increasing the depth—assuming uniform porosity of subsoil—or decreasing the spacing, increases the hydraulic slope of the water table and consequently the rate of movement through the soil and the rate of run-off from the drainage system.

A low water table is necessary only in the planting, growing, and harvesting seasons, and hence in estimating a proper run-off modulus only the storms which occur during the spring and summer months need be considered.

The determination of a run-off modulus is purely a matter of judgment on the part of the engineer, based on his own observations and on the published observations of others of the performance of existing systems, and upon a careful analysis of daily rainfall records at near-by stations. The amount of rainfall in a day is not so important as its intensity. During a hard, dashing rain, the water falls faster than the soil can absorb it, and a large part flows over the surface to the open ditches; on the other hand, the same amount of water precipitated more slowly will all be taken up by the soil and the total run-off from the underdrainage system will be much greater than in the former case. The following example will serve to illustrate this fact. During the summer of 1914, the Iowa Experiment Station conducted a series of run-off investigations in southern Iowa. The soil was originally rather close, but it had become much more open following several years of underdrainage. Quoting from the report of the investigation:

> From early spring until the latter part of May the soil was very dry, though it had been wet when the crop, oats, was planted. The rainfall during June was as follows: June 10, 0.50 inches; June 13, 2.12 inches; June 22, 0.50 inches; July 6, 1.25 inches; July 12, 2.06 inches; and July 16, 1.0 inch. Each of these rains was a heavy downpour lasting but a short time. The water reached the surface so rapidly that a very large proportion of it passed away as surface run-off. So small a portion of this water entered the soil that at no time during June or July was there any flow in the outlet mains of these systems. If the rate of precipita-

tion in these storms had been slow it is very possible that after the rains of July 12 or 16, the outlet main would have been flowing full.

It is not economical to design a system for the run-off from the exceptional storm which may occur once in 10 or 15 years. The ordinary maximum storm should be the basis. Since the ordinary maximum rate of rainfall over a given territory is fairly constant, the run-off would also be constant were it not for the texture of the soil and the topographic features. The difference in the soils can be provided for by varying the spacing and depth of drain; but the difference in surface slopes will necessitate a varying run-off modulus.

It is evident, then, that the proper run-off modulus for a specific section of the country can be stated in general terms only. As an aid to the young engineer in choosing a run-off modulus for a definite area, the following general values are given: ¼ inch for areas with an annual rainfall under 30 inches, such as Minnesota, South Dakota, Nebraska, and Kansas; 5/16 to ⅜ inch for areas with annual rainfall between 30 and 40 inches, such as Iowa, Wisconsin, Michigan, Illinois, Indiana, and Missouri; ½ inch for areas having an annual rainfall between 40 and 50 inches, such as Arkansas, northern Mississippi, Tennessee, Alabama, Georgia, and the Carolinas; and ¾ inch for areas having over 50 inches of annual precipitation, such as Louisiana, southern Mississippi, and Florida. The cost and the efficiency of a drainage system are controlled largely by the choice of a drainage modulus, and, therefore, careful thought should be given to this important factor.

There is little data on the drainage run-off from irrigated areas. The problem here is different from that in humid regions, where the precipitation occurs over the entire drainage area at the same time and in practically the same quantities. Some portions of an arid area may not be irrigated at all, others may be irrigated only a few times during the year, while still others may be irrigated at regular intervals during the growing season. There may, also, be seepage water to remove as well as the excess water applied to the area itself, and this complicates the problem. Excessive irrigation will increase the drainage run-off. Assuming that proper irrigation methods are employed, the amount of excess water to be removed by the tile drains will in most instances be less than 25 per cent of the amount applied in irrigation.

Grades.—The grades of laterals are controlled largely by the surface slope. The laterals are located approximately parallel to the lines of steepest slope so as to obtain the maximum grade for the drains. Since a uniform depth of lateral is desirable, the drains are laid parallel to the average surface slope, provided that the grade thus obtained is not too low. The minimum grade for 4-inch tile should be 0.2 per cent in clayey soils and 0.3 per cent in sandy soils. Where the ground surface is level, the drain cannot be laid at a uniform depth. In this case the laterals are made short, the minimum grade is used, and the middle of the line is placed at the desirable depth. Generally the grade of a single lateral is uniform throughout, since this arrangement usually gives a more uniform depth of tile which is desirable from an agricultural viewpoint. From the viewpoint of hydraulics, however, the grade of the drain should increase from the lower to the upper end, so as to secure a more uniform velocity of flow. In fact, the velocity at the upper end should be greater than at the lower end to offset the decrease in depth of flow—and thereby the transporting power of the flowing water—at the upper end.

The grade of a submain is fixed by the elevations of the lower ends of the laterals emptying into it. These elevations are plotted on profile paper and a grade line established which will pass through or below the points. It may be necessary to modify the grades of the laterals slightly to secure a better grade for the submain. Since the tiles used for the submain are larger than those of the laterals, and the discharge is greater, the danger of silting is less and a flatter minimum grade can be used. A desirable minimum grade for submains is 0.15 foot per 100 feet, although this value will vary somewhat with the size of tile used.

The grade of the main is determined by the elevations of the ends of the submains entering it, and upon the elevation of the outlet into the open channel. Mains above 12 inches in diameter may be laid on a grade as low as 0.05 foot per 100 feet, but steeper grades are desirable.

Size of Tile.—The factors which determine the size of tile to be used for any drain are: (1) the discharge to be carried; (2) the fall or grade; and (3) the resistance to flow offered by the inside surface of the tile.

Discharge.—The discharge is computed from the chosen run-off modulus and the area served by the drain. The modulus in inches of depth per 24 hours is first converted into cubic feet per

second per acre. A run-off modulus of ¼ inch requires a discharge of 0.0105 cubic feet per second per acre; ⅜ inch, 0.0157 cubic feet per second per acre; ½ inch, 0.0210 cubic feet per second per acre; and other values are given in Table XIII. The discharge in cubic feet per second per acre multiplied by the area in acres which each drain serves gives the total discharge which the tile must carry.

Grade.—The method of determining the grade for each line of tile has already been explained.

Roughness Coefficient.—This factor has been fully discussed in Chapter VI (page 152). If Kutter's formula is used in computing the velocity of flow through the drain, a value of n equal to 0.011 or 0.012 should be used. In the U. S. D. A. formula (page 151) the roughness factor is 138.

Computation of Size.—The above three factors being known, the size of tile is determined by a trial process from one of the formulas given in Chapter VII, or from a diagram based on one of them. The size of mains and submains are computed in this way, but not the laterals. Either 4- or 5-inch tiles are universally used for laterals, with the adoption of the 5-inch size growing in favor. The cost of laying a 5-inch tile is no greater than that of a 4-inch tile, and the cost of the tile itself is only slightly greater. To offset this additional cost, the capacity of the larger tile is about 75 per cent greater than that of the smaller. Even the 4-inch size is, however, amply large to remove the soil water for the widest spacing of laterals and the largest run-off modulus, provided the tile is laid true to grade and alignment and *does not become clogged with silt.* Because of these dangers, the use of 3-inch tile has been discontinued, and the 4-inch size is the smallest standard size now manufactured. In irrigated regions, the smallest size is generally 6 inches, because of the wider spacing and greater perviousness of soil.

Illustrative Examples.

1. *Given.*—Area to be drained = 300 acres.

   Grade of main = 0.15 per cent.
   Run-off modulus = ⅜ inch.

   Required.—Size of main.

   Solution.—Discharge = 300 × 0.0157 = 4.71 cubic feet per second. From inspection of diagram in Fig. 72, it is seen that the nearest standard size of tile is 18 inches; hence, use 18-inch tile for main.

2. *Given.*—Twelve parallel lines of 4-inch laterals discharging into a submain approximately at right angles. The laterals are 1,200 feet long, spaced 80 feet apart, and laid on a 0.3 per cent grade. The submain is to be on a grade of 0.2 per cent. The run-off modulus is ¼ inch.

Required.—The size of the submain.

Solution.—Each lateral drains an area of 80 by 1,250 feet, or 2.29 acres. The discharge of each lateral = 2.29 × 0.0105, or 0.023 cubic feet per second. From diagram the discharge of a 4-inch tile on a 0.2 per cent grade is seen to be 0.10 cubic feet per second; hence, the upper 235 feet of submain can be of 4-inch tile. From this point, a 5-inch tile can be used for 320 feet to just above lateral No. 8. A 6-inch tile will be needed from this point to the outlet of the submain.

Junctions.—One tile drain should never enter another at right angles. At the junction of a lateral with a submain, a special junction tile or Y-branch should be used in which the angle between the branches is 45 degrees or less. By the use of a Y-branch, the currents in the two lines of tile unite with the least interference.

If the laterals are laid out at an angle greater than 45 degrees with the submain, the last 15 to 25 feet of the laterals should be curved to meet the submain at the proper angle to connect with the Y-branch. In making this curve, the tile will have to be cut on the inside edge so as to make tight joints. The tile can be chipped with a hammer, judiciously used, or with a monkey wrench. Openings at the top and sides greater than ⅛ to ¼ inch should be covered with a piece of tile of the same diameter to keep out the dirt.

The smaller tile should enter the larger at the middle, and not at the top or the bottom. Also the grade of the last 10 or 20 feet of the lateral should be increased to overcome the additional resistance offered by the curved line of tile, and to prevent a deposit of silt at the junction.

If Y-branches are not available, the connection can be made in the field by cutting an opening in the larger tile with a chisel and hammer and by fitting the lateral tile to it. This practice is not advisable as it is very difficult to get a good connection in this way.

Outlet Structures.—Most drainage systems have their outlets in natural or artificial channels, and the ends of the drains are subject to erosion, undermining, and trampling of live stock, unless properly protected. To prevent injury from these sources, the end of the main should be protected by a concrete headwall

built around the end of the last tile, as shown in Fig. 81. The headwall should be built as soon as the tile trench is ready for the tile so that the first tile can be embedded in the concrete. The foundation should extend below the bottom of the outlet ditch to prevent undermining, and an apron or floor of concrete should be provided for the water from the tile to fall upon. Also, the first 10 or 15 feet of the drain should be constructed of cast-iron pipe or of vitrified sewer pipe with cemented joints.

The mouth of the drain should be protected to prevent small animals from entering the tile. For large drains, iron gratings can be used; for small drains, copper or galvanized-iron screen-

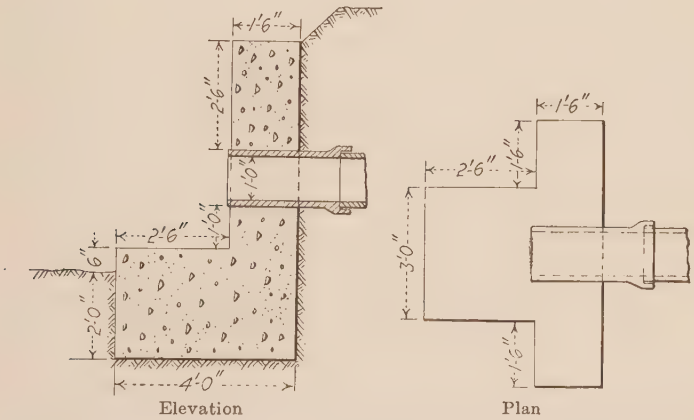


FIG. 81.—Design for concrete headwall for 12-in. tile outlet.

ing. If surface inlets are used in the tile system, a hinged cover should be used so that any débris washed in though the inlets can get out.

Surface Inlets and Catch Basins.—In the more rolling areas where the surface run-off is considerable, due either to the slope of the surface or to a tight subsoil near the surface, surface inlets and catch basins at proper intervals are necessary to prevent surface erosion; also in depressions, without outlet, where surface water collects in large quantities.

Surface inlets are constructed in two ways: One plan consists of an inlet at the top of a catch basin to one side of the tile drain and connected to it by a line of tile; the other plan consists of an inlet at the top of a vertical pipe directly above the tile drain and

connected to it by a T-branch in the top of the drain. In the first plan, sand and silt washed through the inlet collects in the bottom of the catch basin and does not enter the drain, provided the deposit is removed before it reaches the height of the outlet tile. In theory this is the better plan, but owing to the fact that catch basins are rarely cleaned out, the advantage of this type of construction is lost, and it becomes no better than the second plan and is much more expensive to construct. For this reason engineers usually adopt the latter plan.

Inlets are located usually in fence lines where they will not interfere with farming operations and are better protected than in an open field. They are constructed of brick, cement, or several lengths of sewer pipe, and are generally circular, though where concrete is used a square or rectangular shape is easier to construct. Catch basins should be about 4 feet in diameter or section where brick or concrete is used, and 2 feet in diameter when constructed of sewer pipe. The bottom of the basin should be at least 2 feet below the bottom of the tile drain and lined with brick or concrete. A catch basin constructed of 24-inch sewer pipe is more difficult to clean and requires cleaning more often than one of brick or concrete because of its smaller size. Where brick or pipe are used, the joints should be carefully cemented to prevent the entrance of silt.

The surface inlet consists of an iron grating in the top of the catch basin which permits the water to enter, but prevents the entrance of grass, cornstalks, and other débris which might choke the tile drain. Broken rocks or coarse gravel on top of the grating will strain out small trash which otherwise might get into the catch basin. If it is necessary to locate a catch basin in an open field, it should either be protected by a fence or constructed with its top about 1 foot below the surface and covered with rock or gravel.

When surface inlets are used, the tile drain must be designed to remove both the underdrainage and the surface drainage, if both occur at the same time. The amount of surface run-off will, of course, vary with the slope, tributary area, perviousness of soil, and other local conditions. The surface water will reach the drain first and most of it may flow off before the soil water reaches the tile in any considerable quantity. In this case, the capacity of the drain need be increased only slightly. In

extreme cases, the drainage modulus due to both sources of water may amount to 1 inch in 24 hours.

Large Tile Drains versus Small, Open Ditches.—The practice of using large, tile drains as outlets instead of small, open ditches is rapidly growing in favor both with engineers and landowners, especially in those localities where the land is very valuable and where the value of permanent drainage is recognized. Formerly, large tiles were not available, but now sizes up to 60 inches in diameter are standard. Many open ditches are now in use where the discharge could be carried just as readily by tile from 24 to 42 inches in diameter.

The only advantage of the open ditch is low first cost. The disadvantages are many: Open ditches are unsightly; they occupy valuable land; they make the cultivation of farms more difficult; they require bridges every quarter-mile or so; they grow weeds and willows which must be cut annually; they must be cleaned out every few years; and in many cases they require fencing. The elimination of these disadvantages is believed by many to more than offset the greater first cost of large tile drains, and in many cases there can be no doubt of the economy of the latter type of outlet. Where the discharge is such that it can be carried by a large tile, the engineer should at least consider the adoption of the covered outlet in preference to the open one.

Relief Ditches.—Where large tiles are used in preference to an open ditch and the capacity of the tile is not sufficient to remove the run-off during exceptional storm periods which may occur every few years, a relief or surface ditch over the tile drain should be provided. Such ditches are broad and shallow—not over 2 feet deep, and broad enough so that the entire area can be cultivated. Since large, tile drains frequently follow the general line of an old, natural channel or depression, little excavation is usually necessary. The surface ditch should be graded so that all the surface water will run off. The ditch should not discharge into the outlet stream over the headwall of the tile-drain outlet; but a separate channel and outlet should be provided near the tile outlet.

Vertical Drainage.—Where a tight subsoil is underlaid by a gravel or sand formation, an opening through the subsoil to the pervious stratum beneath may furnish an outlet for the water in the surface soil. This is known as vertical drainage. Either

a number of small wells, each draining a small circular area, may be constructed—in which case horizontal lines of tile are dispensed with—or one large well may serve as the outlet of a lateral system of drainage. The small wells are constructed by boring holes with a large auger to the sand formation, and lowering tile into the holes to form vertical columns extending to within 2 feet of the surface, the top being covered with bricks to keep out the dirt. Large wells are dug and lined with brick or encased with large sewer pipe.

This method of drainage is very uncertain and in general is applicable only to small depressions. Its use is not recommended except in special cases.

Construction Surveys.—After a drainage system has been designed and the location of each tile line plotted on the topographic map, the next step is to stake out these lines on the ground and obtain the depth of cut at each stake, so that the tile can be laid true to line and grade. The instruments required for the survey are transit, level, and steel tape.

Starting at Station 0 at the outlet, substantial stakes are driven flush with the ground every 50 or 100 feet along the line of the drain. When the grade of the drain is less than 0.20 foot per 100 feet, the 50-foot interval should be used, as it is very essential that there be no sags in the drain, especially on flat grades. The stakes are not driven over the center line of the drain, but are offsetted so that when the trench is dug the stakes will be at least 1 foot from the edge of the trench. The stakes should either be tacked or else so driven that the center of the stake is not over $\frac{1}{4}$ inch out of line. A guard stake of sufficient length to be seen readily, and with the station number and designation of line marked upon it, should be driven about 6 inches outside of each line stake. A line stake should be placed also at each point where a submain enters a main and a lateral enters a submain; and the fact that the stake marks a junction point should be clearly indicated on the guard stake. The last stake on each line should be marked "end."

The directions of the several lines are determined in the field by measurements scaled from the map, taken from definite points in the field, such as former survey stakes, fence corners, trees, or other objects. A change of direction in any line should be made by a uniform curve and never by a sharp angular turn. A radius of 50 or 100 feet should be used, and stakes placed at

25-foot intervals. Points on a curve can be located either by the deflection angle method used in railroad and highway work, or by locating the center of the curve and striking an arc on the ground, using the tape as a radius and driving as many line stakes as desired.

After the line stakes have been located, the cut at each stake must next be found. Before starting the level work, the elevation of the bottom of the tile trench at each line stake should be computed from the profiles and checked. These elevations are recorded in the level notebook opposite the corresponding stations as shown in Fig. 82. Using a bench mark established during the

LEVELS FOR TILE DRAINS ON FARM OF T. C. JOHNSON

Station	Grade elevation	Differential levels	Rod reading	Ground elevation	Cut, feet
15	687.17	696.24	5.21	691.03	3.86
+50	687.27	−4.65	5.30	690.94	3.67
16	687.37	691.59	4.96	691.28	3.91
+50	687.47	+5.27	4.65	691.59	4.12
17	687.57	696.86	5.40	691.46	3.89
+50	687.67		5.45	691.41	3.74

FIG. 82.—Sample form of level notes.

former survey, profile levels are then run over each line, and the elevation of the top of each line stake determined to the nearest 0.01 foot. The computed grade elevation subtracted from the measured ground elevation gives the depth of trench at each stake. For the convenience of the contractor it may be necessary to convert these cuts in feet and decimals of a foot into feet and inches, to the nearest ⅛ inch. These cuts are marked on the guard stakes, and in addition a memorandum of the stations and cuts is prepared for the contractor. A very satisfactory form of level notes is shown in Fig. 82.

Recording Plans.—Mains and submains are frequently constructed by drainage districts and the plans and profiles are recorded in accordance with the drainage law. The minute

farm drainage, however, is done upon the initiative of the individual owners, and no official record is automatically kept of the location of the drains or of the amount of drainage accomplished. Frequently, the investment in drainage is nearly as much as the investment in land, and it is very much to the interest of the landowner to have an official record to prove the amount of drainage work done. It is a very simple and inexpensive matter to have the plans and profiles recorded in the County Recorder's office, together with the affidavits of the engineer and contractor; and the improvement should not be considered completed until this has been done.

Tile.—*Kinds.*—As regards material of manufacture, drain tiles are of two kinds—clay and concrete. Clay tiles are made of shale, fire clay, or surface clays; and concrete tiles of a suitable mixture of Portland cement, mineral aggregate, and water, hardened by hydraulic chemical reaction.

Classes.—Both clay and concrete tile are divided into three classifications, based on strength. These are, *Farm Drain Tile, Standard Drain Tile,* and *Extra-quality Drain Tile.* Farm drain tiles are suitable for ordinary private drainage work on farms for moderate sizes and depths; standard drain tiles, for ordinary district land drainage at moderate depths; and extra-quality tiles, for district land drainage at considerable depths and where extra quality is desired.

Sizes.—The sizes of drain tiles are designated by interior diameters. The standard sizes as recommended by the American Society for Testing Materials are as shown in Table XXXVI, and in addition thereto sizes 45, 48, 54, and 60 inches. These are omitted from the table for want of experimental data on strength requirements. After a careful study of the subject, Committee C-6 on Drain Tile of this Society—which is composed of both engineers and manufacturers—came to the conclusion that sizes other than those listed in the table were not necessary in drainage work, and that in the interest of standardization and economy, their manufacture should be discontinued. The tentative revision of the Standard Specifications in 1923 contains the above recommendation. At present, however, in addition to the above sizes, the even 2-inch sizes up to 42 inches are available.

The Standard Specifications require that drain tile smaller than 12 inches shall be at least 12 inches in length; tile from 12 to

30 inches in diameter, inclusive, shall have lengths not less than their diameters; and tile larger than 30 inches in diameter shall have a minimum length of 30 inches. As generally manufactured, the sizes up to 8 inches are made in 1-foot lengths; sizes 8 inches to 12 inches, from 1.5 to 2 feet; sizes 12 inches to 24 inches, 2 to 2.5 feet; and above 24 inches, 3 feet in length.

TABLE XXXVI.—PHYSICAL TEST REQUIREMENTS FOR DIFFERENT CLASSES OF DRAIN TILE

Caution.—Drain tile meeting these requirements are not necessarily safe against cracking in ditches. For safe requirements as against cracking see Table XXXVIII.

Internal diameter of tile, inches	Farm drain tile				Standard drain tile				Extra-quality drain tile			
	Minimum average ordinary supporting strength, pounds per linear foot	Maximum average absorption by standard boiling test, per cent			Minimum average ordinary supporting strength, pounds per linear foot	Maximum average absorption by standard boiling test, per cent			Minimum average ordinary supporting strength, pounds per linear foot	Maximum average absorption by standard boiling test, per cent		
		Shale and fire-clay tile	Surface-clay tile	Concrete tile		Shale and fire-clay tile	Surface-clay tile	Concrete tile		Shale and fire-clay tile	Surface-clay tile	Concrete tile
4	800	11	14	12	1,200	9	13	10	1,600	7	11	9
5	800	11	14	12	1,200	9	13	10	1,600	7	11	9
6	800	11	14	12	1,200	9	13	10	1,600	7	11	9
8	800	11	14	12	1,200	9	13	10	1,600	7	11	9
10	800	11	14	12	1,200	9	13	10	1,600	7	11	9
12	800	11	14	12	1,200	9	13	10	1,600	7	11	9
15	1000	11	14	12	1,300	9	13	10	1,600	7	11	9
18					1,400	9	13	10	1,800	7	11	9
21					1,550	9	13	10	2,100	7	11	9
24					1,700	9	13	10	2,400	7	11	9
27					1,850	9	13	10	2,700	7	11	9
30					2,000	9	13	10	3,000	7	11	9
33					2,150	9	13	10	3,300	7	11	9
36	(Not permitted)	(Not permitted)	(Not permitted)	(Not permitted)	2,300	9	13	10	3,600	7	11	9
39					2,450	9	13	10	3,900	7	11	9
42[1]					2,600	9	13	10	4,200	7	11	9

[1] Larger sizes recommended as standard are 45, 48, 54, and 60 inches.

Weight.—The length of tile, approximate weight per foot, and the number of linear feet in a carload of 15 tons, as listed by one well-known manufacturer of clay tile, are given in Table XXXVII.

TABLE XXXVII.—DRAIN TILE DATA

Inside diameter, inches	Length of tiles, feet	Approximate weight per foot, pounds	Carload (15 tons), feet
3	1	5	6,000
4	1	7	4,300
5	1	9	3,400
6	1	11	2,800
8	2	22	1,400
10	2	27	1,150
12	2½	36	850
14	2½	44	700
15	2½	52	600
16	2½	57	550
18	2½	72	425
20	2½	95	325
22	2½	105	300
24	2½	118	250
27	3	178	170
28	3	200	150
30	3	215	140
33	3	250	120
36	3	295	100
39	3	375	80
42	3	425	75

Strength.—The engineer is concerned not so much with the material of which the tile is made as he is with its strength and durability. Most specifications for tile contain a clause stating that the tile shall meet the physical test requirements given in the Standard Specifications of the American Society for Testing Materials. These requirements as revised in 1923 are given in Table XXXVI. Standard methods of making the physical tests are contained in the Standard Specifications and the engineer should be familiar with them.

In case of doubt as to the ability of the tile to resist the chemical action of soil waters of marked acid or alkaline character, special chemical requirements may be included in the specifications. Hard-burned clay tiles do not seem to be affected by acid or alkaline solutions, but concrete tiles are affected to a greater or less extent by solutions of sodium and magnesium sulfate, especially the latter. This subject is now under investigation by four separate agencies—the Iowa Engineering Experiment Sta-

tion, the Structural Materials Research Laboratory of Lewis Institute, the United States Bureau of Public Roads, and the Engineering Institute of Canada. The results of these investigations to date are reported in the 1923 report of Committee C-6 of the American Society for Testing Materials.

Inspection.—All tiles should be thoroughly inspected at the trench to determine whether they comply with the specifications of general properties, independent of the physical and chemical test requirements. These general properties are:

1. Drain tile should be approximately straight and circular in cross-section.
2. The ends should be so regular and smooth as readily to admit of making close joints.
3. They should be substantially uniform in structure throughout. Examination can be made of fractured surfaces.
4. Clay tile should give a clear ring when stood on end and, while dry, tapped with a light hammer. Concrete tile will not give as clear a ring as clay tile.
5. The tile should be reasonably smooth on the inside.
6. The tile should be free from cracks and checks extending into the body of the tile in such a manner as to decrease the strength materially.

When physical tests are not made, because of the expense involved, the inspection should be especially rigid, since a few weak tiles may endanger the successful operation of the entire system. Without doubt it pays to buy good tiles and the best safeguard is to buy them from a manufacturer of well-established reputation. When concrete tiles are to be used, the landowner should not attempt to make his own tile, as it cannot be done successfully.

CONSTRUCTION OF TILE DRAINS

Trenching.—Tile trenches may be dug either by hand or by machinery. Where the amount of trenching is large and especially if the trenches are deep, machine work may prove more satisfactory and economical than hand work. The trenches for laterals, however, are generally dug by hand.

First, a guide line is stretched on the ground along one edge of the trench, as illustrated in Figs. 83 and 84, so that it will be started straight. The trench should be as narrow as possible so as to reduce the load on the tile. A width of 12 inches is

sufficient for laterals up to about 36 inches in depth, and 16 inches in width for greater depths. For larger tile, the trench will have to be wider, but only wide enough to provide sufficient space between the tile and the sides of the trench to make it possible to tamp thoroughly the back-filling around the pipe. About one-third the nominal diameter of the tile on each side will generally provide sufficient working space. For large tile, the maximum width of trench should be specified.

The trench is dug with an 18-inch tile spade, one course at a time, as illustrated in Figs. 83 and 85. Before the last course is dug, a gage line is set over the center of the trench at a convenient distance above the surface and parallel to the bottom of the trench. This is done by driving substantial stakes vertically on each side of the trench at each grade stake and nailing a crossbar horizontally to each pair of stakes, as shown in Fig. 91. These crossbars, or grade bars, are established at a uniform distance above the bottom of the trench, usually 6 feet. Then 6 feet minus the cut at the grade stake is the distance the grade bar is set above the top of the grade stake. A nail is driven into the top of the grade bar to mark the center line of the drain. A strong, light line, or small wire, is then stretched tightly across the tops of the grade bars at the nails (see Fig. 92). By using a 6-foot grade rod, the workmen can readily tell when the trench is to the proper depth. The last spading is taken to within only 1 or 2 inches of the finished grade, and the bottom brought to grade with a tile-scoop which leaves the bottom of the trench curved to the shape of the tiles, thus giving a wider bearing surface on the bottom and preventing the tiles from rolling out of line when dirt is placed upon them. The tile-scoop is operated from the surface for narrow, shallow trenches. The depth of trench should be tested for every length of tile to insure a uniform grade.

The best time to lay tile in ordinary soils is in the late spring when the ground is in the best condition for digging. For marsh and swamp land, late summer is a better time.

When a machine is used for the trenching, the grade bars are set in advance of the machine and at the same distance above grade as the "sighting point" on the machine. It is usually necessary to finish by hand the trench made with a machine. Some of these machines not only dig the trench but also lay the tiles and back-fill the trench. It is not advisable to use a machine

FIG. 83.—Digging the first spading true to a line.

FIG. 84.—Cleaning out after first spading.

NOTE.—Figures 83 to 90 are reproduced here through the courtesy of the Mason City Brick & Tile Co., Mason City, Iowa.

FIG. 85.—Digging the second spading.

FIG. 86.—Shaping bottom to receive the tile.

FIG. 87.—Laying the tile.

FIG. 88.—Ready to cover.

FIG 89.—Blinding the tile.

FIG. 90.—Tile is covered and the end tile plugged to keep out dirt and animals.

for laying the tiles, since it is difficult, if not impossible, to secure good results in this way.

FIG. 91.—Setting grade-bars.[1]

FIG. 92.—Using grade-bars to get correct grade of trench.[1]

Laying the Tiles.—The tiles are laid upgrade, beginning at the outlet. Each tile should be laid true to line and grade and firmly bedded in the bottom of the trench. As they are laid, the tiles

[1] Bull. 234, North Carolina Experiment Station.

are turned to give the tightest joint possible, especially at the top. In sandy soils, the widest crack between the joints should be ⅛ inch; in clay soils, ¼ inch. Openings wider than these should be covered with pieces of broken tile. Muddy water in small quantities should be kept out of the tile to prevent silting. Small tiles in shallow trenches with firm bottoms are laid generally from the surface, the tiles being fitted on the surface and lowered into the trench in the proper position with a tile hook as illustrated in Fig. 87.

When the bottom of the trench is soft, great difficulty is experienced in laying the tiles. Sometimes the soft material can be removed and replaced with good clay or loam soil; sometimes it is necessary to lay the tile on narrow boards supported on piles; again, in the case of large tiles, it may be necessary to embed the tile in a concrete base 2 or 3 inches thick. In fine sand, the joints should be wrapped with burlap or building paper to prevent the fine material from entering the drain.

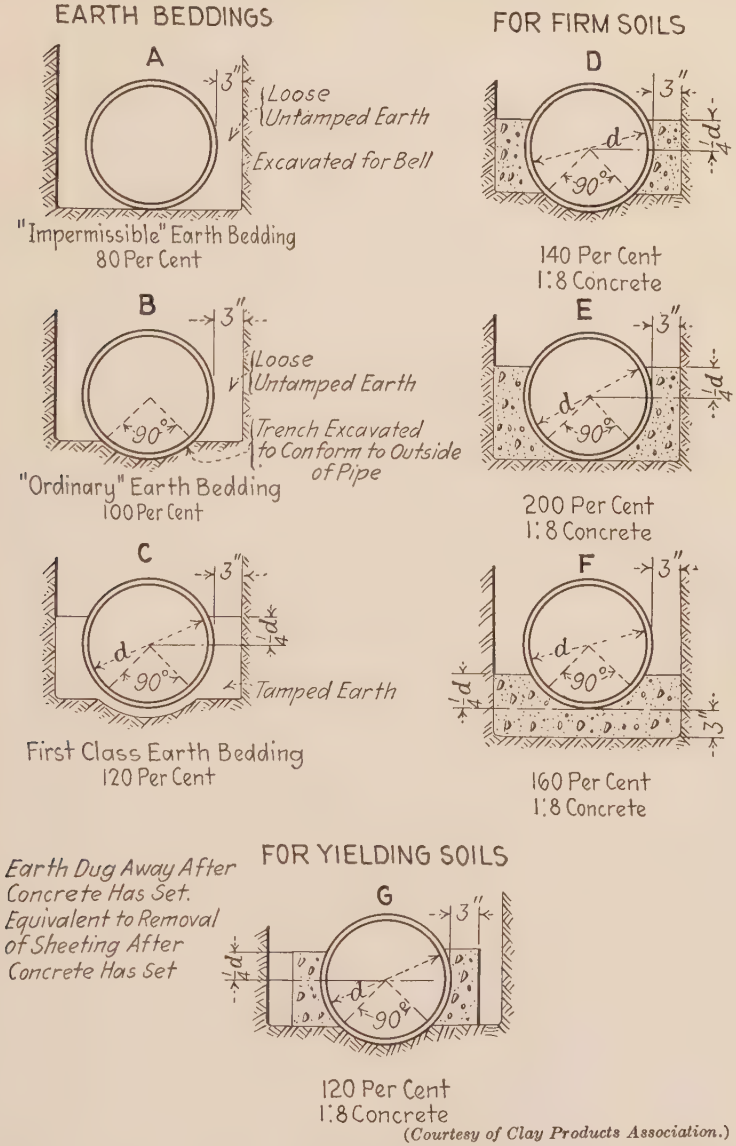
The end of a tile drain should always be tightly closed with brick or pieces of tile to keep out the dirt.

Blinding the Tiles.—After the tile-laying has been carefully inspected to insure that the work has been properly done, the tiles are "blinded" by covering them to a depth of 6 to 12 inches with loose, mellow topsoil shaved from the sides of the trench with a spade as illustrated in Fig. 89. Sandy soil should never be used for this purpose. The loose soil falling on the tiles in small quantities will not move them out of line, and this layer of material will act as a cushion to prevent damage when the rougher back-fill is made. In the case of the larger sizes, earth should be well packed around the tiles.

Back-filling.—After the tiles have been blinded, the remainder of the trench can be filled in the most convenient manner. This is usually accomplished by shovels, team and plow, or some form of scraper.

Supporting Strength of Drain Tiles.—The load imposed upon drain tiles by the back-filling material depends upon the breadth of the trench at the top of the tiles, the depth of fill above the top of the tiles, and the nature of the material. It is to be noted that the breadth of trench rather than the size of the tiles in the trench determines the load on the tiles. Because of this fact, trenches should be as narrow as is consistent with good workmanship.

In order to specify tiles of sufficient strength, the engineer must know, approximately, the magnitude of the load which will come



(*Courtesy of Clay Products Association.*)

FIG. 93.—Methods of bedding tile.

on the tiles. Table XXXVIII, taken from the Standard Specifications of the American Society for Testing Materials,

TABLE XXXVIII.—STANDARD ORDINARY SUPPORTING STRENGTHS OF DRAIN TILE FOR ORDINARY SAND AND FOR THOROUGHLY WET CLAY DITCH FILLING MATERIALS

Strengths in Pounds per Linear Foot

Height of fill above top of tile, feet	Breadth of ditch at top of tile: 1 foot				2 feet				3 feet				4 feet				5 feet			
	Method of laying pipe: Ordinary		First-class		Ordinary		First-class		Ordinary		First-class		Ordinary		First-class		Ordinary		First-class	
	Ditch-filling material		Ditch-filling material		Ditch-filling material		Ditch-filling material		Ditch-filling material		Ditch-filling material		Ditch-filling material		Ditch-filling material		Ditch-filling material		Ditch-filling material	
	Sand	Clay	Sand	Clay	Sand	Clay	Sand	Clay	Sand	Clay	Sand	Clay	Sand	Clay	Sand	Clay	Sand	Clay	Sand	Clay
2	265	280	220	235	615	635	510	530	970	990	810	830	1,330	1,350	1,110	1,130	1,690	1,710	1,410	1,430
4	400	450	335	375	1,050	1,120	880	935	1,750	1,820	1,460	1,520	2,450	2,540	2,040	2,110	3,160	3,250	2,640	2,710
6	470	545	390	455	1,370	1,500	1,140	1,250	2,370	2,530	1,980	2,110	3,410	3,580	2,840	2,980	4,460	4,640	3,720	3,870
8	505	605	420	505	1,600	1,790	1,330	1,490	2,870	3,110	2,390	2,590	4,220	4,490	3,510	3,740	5,590	5,890	4,660	4,910
10	525	640	440	535	1,760	2,010	1,470	1,680	3,270	3,610	2,730	3,010	4,900	5,290	4,080	4,410	6,590	7,020	5,490	5,850
12	535	660	445	550	1,880	2,190	1,570	1,820	3,600	4,030	3,000	3,355	5,480	6,000	4,570	5,000	7,460	8,030	6,220	6,690
14	540	675	450	560	1,960	2,320	1,640	1,930	3,850	4,380	3,210	3,650	5,980	6,620	4,980	5,520	8,230	8,950	6,850	7,460
16	545	680	455	565	2,030	2,420	1,690	2,020	4,060	4,670	3,390	3,890	6,400	7,160	5,330	5,970	8,890	9,770	7,410	8,150
18	545	685	455	570	2,070	2,500	1,720	2,090	4,230	4,920	3,530	4,100	6,750	7,630	5,630	6,360	9,480	10,520	7,900	8,770
20	545	690	455	575	2,100	2,560	1,750	2,140	4,370	5,130	3,640	4,280	7,050	8,060	5,880	6,720	9,990	11,200	8,330	9,330
22	545	690	455	575	2,120	2,610	1,770	2,180	4,470	5,310	3,730	4,420	7,310	8,430	6,090	7,020	10,400	11,800	8,700	9,830
24	545	690	455	575	2,140	2,650	1,780	2,210	4,560	5,450	3,800	4,540	7,520	8,750	6,270	7,290	10,800	12,300	9,030	10,300
26	545	690	455	575	2,150	2,670	1,790	2,230	4,630	5,580	3,850	4,640	7,700	9,030	6,420	7,530	11,200	12,800	9,320	10,700
28	545	690	455	575	2,160	2,700	1,800	2,250	4,680	5,680	3,900	4,740	7,860	9,280	6,550	7,740	11,500	13,300	9,570	11,100
30	545	690	455	575	2,170	2,710	1,810	2,260	4,730	5,770	3,940	4,810	7,990	9,500	6,660	7,920	11,800	13,700	9,800	11,400
Very great.....	545	690	455	575	2,180	2,770	1,820	2,310	4,910	6,230	4,090	5,190	8,730	11,100	7,270	9,230	13,600	17,300	11,400	14,400

NOTE.—Ordinary pipe laying is pipe laying in accordance with customary good practice in tile-drain construction, whereby the underside of the pipe is well bedded on soil for 60 to 90 degrees of the circumference.

First-class pipe laying is pipe laying in accordance with the best customary practice in pipe-sewer construction, whereby the entire underside of the pipe is very thoroughly bedded on soil and the entire pipe is surrounded by well-compacted soil, under the direction of an inspector constantly present on the work.

When pipe is laid in a concrete or other permanent masonry cradle, strong enough to carry the entire load to the subbase without breaking and large enough to prevent material settlement, the standard strengths for all dimensions of ditches and all filling materials shall be those specified for standard drain tile in Table XXXVI.

gives the standard ordinary supporting strengths of drain tiles for ordinary sand and for thoroughly wet clay ditch-filling material, for various breadths of trench and depths of fill. For example, any size of drain tile laid in accordance with customary good practice in tile-drain construction in a trench 2-feet wide, and with 4 feet of clay-filling material above the top of the tile, will be subjected to an approximate pressure of 1,120 pounds per linear foot. Table XXXVI shows that farm drain tile will not withstand this pressure, and hence standard drain tile will be required. If the depth of filling over the tile is 6 feet—the other conditions remaining the same—the ordinary supporting strength of the tile will be approximately 1,500 pounds per linear foot and extra-quality drain tile will be necessary.

The strength of the tile can be increased by proper methods of bedding as shown in Fig. 93. The ordinary method of laying referred to in Table XXXVIII is shown in Fig. 93*b*; and the first-class method of laying, in Fig. 93*c*.

Cleaning Tile Drains.—As the soil water enters the drain, it carries with it more or less silt, the amount depending upon the nature of the soil around the tiles and the size of the joints between them. If the volume and velocity of the water flowing through the tiles are not sufficient to transport this silt, it will be deposited in the drain and in time will obstruct the flow to such an extent as to make its removal necessary. This condition rarely occurs when the fall is ample and the tiles properly laid.

To clean a tile drain, it is customary to uncover a few feet of drain at intervals of about 25 feet, remove several tiles, and clean between the openings by forcing some form of swab or silt remover through the drain. If the material is soft, a jointed rod made up of 3.5 foot lengths is first used to push a rope through the drain. A swab made of a canvas bag stuffed with straw is then drawn back and forth until the drain is clean. The material removed should be shoveled out of the trench. If the material is too solid to be removed by this type of swab, a more rigid device must be used, such as a cylindrical wire brush or a substitute of home manufacture. In extreme cases, the material may have to be scooped out with a long-handled scoop or a hoe with a hinged blade.

Sometimes a drain becomes obstructed at some one point due possibly to the settling or collapse of the tiles. The point of obstruction is located by digging openings similar to those already mentioned and noting whether the water rises in the hole; if so,

the obstruction is below the opening. Large tiles seldom become filled with silt unless the tiles collapse. This happens too frequently with tiles over 12 inches in diameter and is caused by the cracking of the tiles under the weight of the trench-filling material, and its collapsing the first time the drain is surcharged.

Clogging of drains may also be caused by the roots of trees entering the joints and filling the tiles. Willow, soft maple, and elm trees are especially troublesome and their roots frequently reach out as far as 100 feet to a drain. Where trouble is anticipated from the roots of near-by trees, either the trees should be cut down or else the joints of the tiles within the danger zone should be surrounded with concrete.

Cost of Tile Drains.—The several items entering into the cost of a tile-drainage system are: (1) the cost of the tiles at the factory; (2) the freight charges on the tiles from the factory to the nearest railroad station; (3) the hauling of the tiles from the station to the area to be tiled, and the distributing of the tiles along the line of the trench; (4) the trenching, tile laying, and blinding; (5) the back-filling; (6) the cost of special structures, such as outlets, catch basins, and surface inlets; (7) engineering and supervision; and (8) contingencies.

Cost of Tiles.—The cost of tiles varies considerably in different parts of the country, and fluctuates at individual factories with the cost of labor and of coal. The cost of either clay or concrete tiles under each of the three standard classifications and the cost of Y's and T's can be obtained upon application to the nearest factory manufacturing a satisfactory product.

TABLE XXXIX.—COST OF SHALE DRAIN TILE IN ILLINOIS
As of Mar. 1, 1924

Size of tile in inches	Cost per 1,000 feet	Size of tile in inches	Cost per 1,000 feet
4	$ 22	16	$ 315
5	32	18	440
6	40	20	520
8	70	22	635
10	100	24	750
12	155	27	1,200
14	265	30	1,610
15	280	32	1,820

The prices of shale drain tiles in Illinois, f.o.b. factory, are given in Table XXXIX.

Freight Charges.—Some factories quote prices f.o.b. factory; others, prices f.o.b. destination on carload lots. In case freight charges are not included, it is a simple matter to determine the freight rates from the agent at the nearest railroad station. The number of carloads required can be computed from the data in Table XXXVII. The manufacturer should be responsible for tiles broken in transit.

Hauling Tiles.—The cost of hauling tiles from the railroad station to the ditch will vary with the length of haul, the conditions of the roads, and the local prices of teams or trucks and labor. Where the hauling is done by contract, the unit of price is usually the ton (2,000 pounds). Owing to the bulk of drain tiles, a ton is generally considered a good wagon load, and the cost of hauling can be estimated on this basis, if more accurate data are not available. The cost of hauling should include the stacking of the tiles in small piles at proper intervals along the trenches.

Trenching, Laying Tiles, and Blinding.—It is customary to include these three items under one contract price, the unit of price being the rod or the foot, and varying for different depths. The cost of this part of the work will depend upon local prices

TABLE XL.[1]—COST OF TRENCHING, LAYING TILES, AND BACK FILLING TRENCHES IN CENTRAL ILLINOIS, AS OF 1924

Prices Are Based on Depth of Trench of 4 Feet

Diameter of tiles, inches	Cost per rod	Diameter of tiles, inches	Cost per rod
4	$0.64	18	$2.60
5	0.68	20	3.00
6	0.70	22	3.30
8	0.92	24	3.96
10	1.08	27	4.30
12	1.20	28	4.80
14	1.40	30	5.00
15	2.00	33	5.50
16	2.25	36	7.50

[1] DAPPERT, J. W., Taylorville, Illinois, "Drain Tile Costs and the Farmer," *National Reclamation Magazine*, p. 13, January, 1925.

of labor and upon the nature of the soil, that is, whether it is easy or difficult to dig, whether sheeting will be required, and whether or not soft material will be encountered at the bottom of the trench. Where the work is let by contract, the prices of local contractors are readily obtainable. When the work is done by day labor, the time required can be estimated from the following data, if more definite data are not available. A good tile ditcher should be able in a day to dig about 140 feet of 3-foot trench in sandy or loamy soils, and lay 4- or 5-inch tiles and blind them; in clay soils, from 80 to 115 feet is a good day's work, depending upon the hardness of the clay; in soft muck or peat soils, about 200 feet should be laid in a day. Table XL gives the current prices (1924) in Central Illinois for trenching, laying tiles, and back-filling trenches.

Engineering.—On small farm systems, the engineer generally charges for his services on a per diem basis, and since the total time required of him is short and the nature of the work rather definite, it is not difficult to estimate fairly accurately the amount of his remuneration. On large projects, however, where the amount of time required is more difficult to estimate and where his responsibility is greater, the present tendency is for the engineer to receive a certain percentage of the total cost of the work. This percentage varies from 4 to 12 per cent depending upon the magnitude of the work. For average projects, 6 per cent can be used in making up the estimate of cost.

Contingencies.—To cover fluctuations in prices of materials and labor, and the possible omission of a minor item in the estimate, an additional 5 per cent of the total cost should be included.

Line	Sta. to Sta.	% Slope	Size	Length
A-2	1100 - 10+50	.174	12"	950'

Line	6"	5"	8"	10"	12"
A-2	100	525'			10'

CHAPTER X

PUMPING PLANTS FOR DRAINAGE DISTRICTS

Drainage districts lying in the flood plains of large streams or along the coast of tidal waters require not only open ditches and tile drains for their reclamation but also levees for protection from inundation by the flood waters of the stream or by the tidal waters at high tide, and, when a gravity outlet for the ditches of the district during high water is not available, a pumping plant is required for lifting the water over the levee.

While it is not necessary that the drainage engineer should be able to design a pump or prime mover for a pumping plant, he should be able to choose intelligently the number and sizes of pumps necessary for the work in hand and the kind and power of prime mover for operating the pumps, to design and write specifications for the building and pumping equipment, to estimate the cost of the same, to superintend the erection of the plant and the installation of the equipment, to test the pumps to see that the specifications have been met, and to superintend the operation of the plant when completed. It is the purpose of this chapter to deal with these subjects as fully as is compatible with the space allotted, and to prepare a foundation upon which the student can build and extend his knowledge of the subject by individual research.

Location of Plant.—The chief factor which controls the location of a pumping plant is topography; other factors are proximity to towns, nature of foundation, and accessibility for delivery of fuel.

Generally, the interior ditch system is first designed in accordance with the topography and with the need and completeness of drainage, and the pumping plant located at the lower end of the main ditch at the lower end of the district; and where the natural surface slope is good, this is the only solution, since it is impracticable for the ditches to run opposite to the natural slope. Even in this event, however, the location of the pumping plant and the main ditch should not be fixed until borings have been made at the proposed site to determine the nature of the foundation

for the building and the machinery. Such borings frequently result in a change of site and a modification of the ditch system. Too much stress cannot be laid upon the importance of a good foundation.

Some districts have their pumping plants at the upper ends of the districts so as to be near a town or city and thus make the plants more accessible. This is possible where the land is level or nearly so, so that the ditches can discharge equally well in either direction.

When steam is used for power, the location of the plant should be such that coal or oil can be delivered to the plant under ordinary conditions.

From the standpoint of operation, it is very desirable that the main ditch at the plant be enlarged to form a reservoir. Occasionally, an old pond or slough at or near the levee will provide a natural reservoir, and the plant may be located at this point to take advantage of this situation.

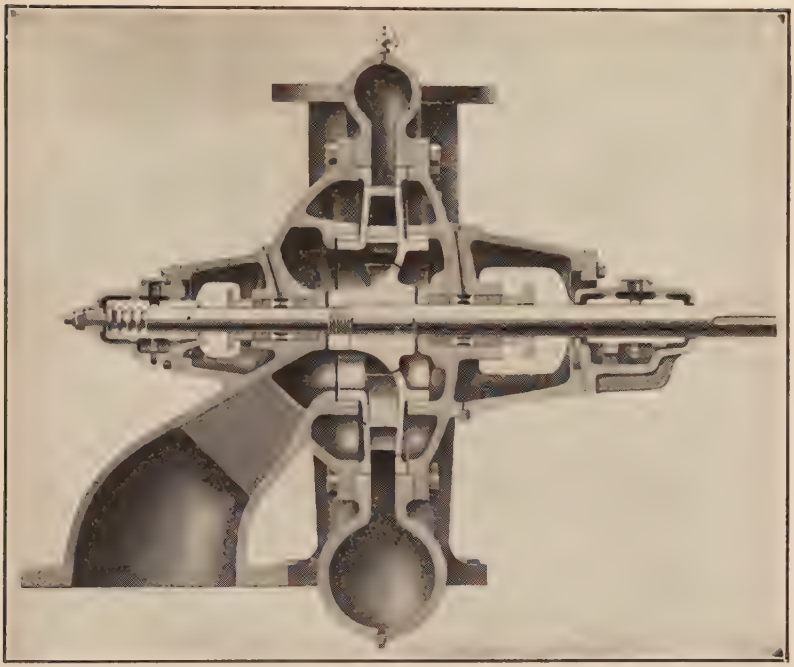
As regards the location of the plant with respect to the levee, it is placed on the inside of the levee and as close to it as possible so as to make the length of the discharge pipe a minimum.

PUMPS AND ACCESSORIES

Kinds of Pumps.—Centrifugal, rotary displacement, plunger, and screw pumps and scoop wheels have all been used for removing drainage water. Of these, the centrifugal pump is generally best adapted to drainage work and is used almost exclusively at the present time. Screw pumps of large size are in successful use in Louisiana. In the larger sizes, the screw pump is cheaper than the centrifugal pump of equal capacity and operates more satisfactorily under changing heads. The centrifugal pump and the screw pump are the only ones which will be considered in this discussion.

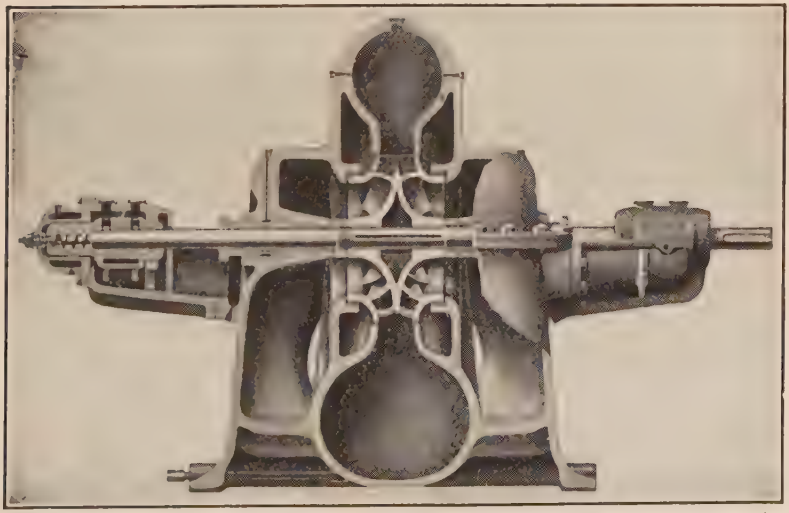
Centrifugal Pumps.—The advantages of centrifugal pumps are that they are cheap, simple in construction, have no valves, occupy little space, can be obtained in all sizes, and are easy to operate. Their disadvantages are that they must be primed in starting, and have a lower efficiency under the conditions which prevail in drainage pumping than any other type of pump.

Description.—The volute type of centrifugal pump, which is used in drainage pumping, consists of an impeller or runner with curved vanes rotating on a horizontal shaft within a spiral case.



(R. L. Daugherty's *Centrifugal Pumps.*)

FIG. 94.—Single suction centrifugal pump.



(R. L. Daugherty's *Centrifugal Pumps.*)

FIG. 95.—Double suction centrifugal pump.

The water enters the center or eye of the impeller through the suction pipe. If the water enters the impeller from one side only, as shown in Fig. 94, the pump is known as a single-suction volute pump. Small pumps are generally of this type. If the water enters from both sides, as shown in Fig. 95, the impeller is double and the pump is known as a double suction volute pump.

As the water flows radially outward through the rotating impeller, it receives energy from the vanes, resulting in an increase both in pressure and in velocity. The spiral casing is so designed that the velocity of the water is gradually reduced as it flows from the impeller to the discharge pipe, thus transforming kinetic energy into pressure.

Size.—By the size of a centrifugal pump is meant the diameter of the discharge-pipe connection in inches. The minimum size of pump used in land drainage is usually 15 inches. The sizes most used are 18, 24, 30, and 36 inches; but practically all sizes are available.

Capacity.—The capacity of a pump of a given size depends upon the velocity of discharge and this in turn depends upon the

TABLE XLI.—CAPACITY OF CENTRIFUGAL PUMPS, BASED ON A DISCHARGE VELOCITY OF 10 FEET PER SECOND

Size in inches	Capacity	
	Cubic feet per second	Gallons per minute
12	7.8	3,500
15	12.3	5,520
18	17.7	7,950
20	21.8	9,800
22	26.4	11,850
24	31.4	14,100
26	36.8	16,500
28	42.8	19,200
30	49.1	22,050
32	55.8	25,050
36	70.7	31,750
39	82.9	37,200
42	96.2	43,200
45	110.5	49,600
48	125.7	56,400
54	159.0	71,380
60	196.3	88,150

speed at which the pump is operated, the design of the impeller vanes, and the head of water pumped against. Manufacturers generally rate their pumps for the head and speed which give the maximum efficiency. The discharge velocity under this condition of operation is usually about 10 feet per second, and in some instances as high as 12 feet per second. These values may be obtained when the proper speed of operation is used for the various heads; but as operated in practice, the discharge velocity is probably nearer 8 than 10 feet per second, and this fact must not be overlooked in choosing the size of pumps. Table XLI gives the discharges in cubic feet per second and in gallons per minute for pumps of various sizes, based on a velocity of 10 feet per second through the discharge-pipe connection.

Characteristics.—The "characteristics" of a pump are the curves showing the relation between head, discharge, speed, and horsepower. They can be plotted in various ways with different factors constant; but generally they are constructed for a constant speed. Figure 96 shows the characteristic curves for a 36-inch centrifugal pump for two constant speeds. The full-line curves are for a constant speed of 200 revolutions per minute and the broken-line curves are for a constant speed of 275 revolutions per minute. From these curves, the performance of the pump can be determined for any condition of operation.

The data for characteristic curves are obtained by testing a pump and finding the discharge and brake horsepower at various heads for the several speeds at which the best efficiency of the pump may be expected throughout the range of total heads. The water or theoretical horsepower can be calculated from formula 37 for the heads and discharges taken from the head-capacity curve. From the brake horsepower curve and the water horsepower curve, the efficiencies can be computed and plotted, giving the efficiency curve.

The full-line curves in Fig. 96 show that when the head is zero, the pump will discharge 37,300 gallons per minute at 200 revolutions per minute. The water horsepower is zero because no useful work is done. The brake horsepower is about 175, that is, it requires 175 horsepower applied at the pump shaft to discharge 37,300 gallons per minute through the pump against friction head only without raising the water at all. At point *A* on the head-capacity curve, the total head is 10 feet, the discharge is 33,500 gallons per minute, the brake horsepower is 170 as shown

at point C, the water horsepower is 85 as shown at point D, and the efficiency, therefore, is 50 per cent, as shown by the efficiency curve at point B. The highest efficiency of the pump is about 78 per cent as shown at point F, when pumping against a total head of 18.5 feet and discharging 22,000 gallons per minute as indicated at point E. The brake horsepower at this point is 132 and the water horsepower is 102 as shown at points G and H, respectively. At point I on the head-capacity curve, the efficiency is again 50 per cent. Thus it is seen that for this pump a

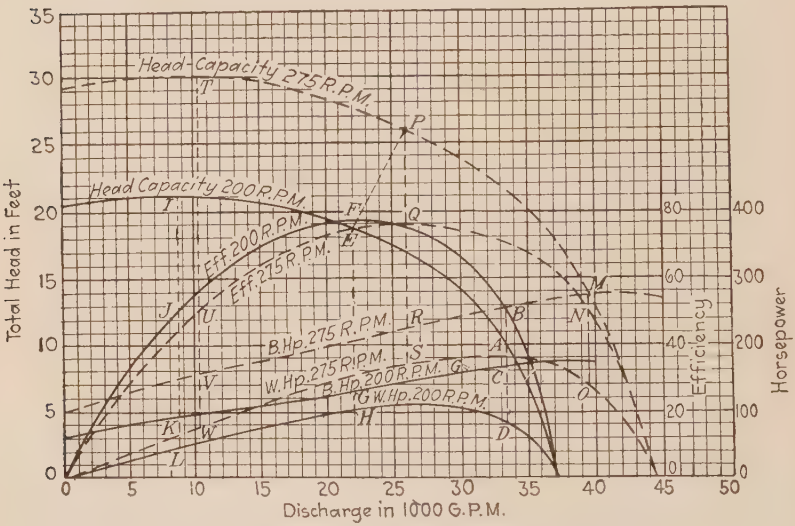


FIG. 96.—Characteristic curves for a 36-in. centrifugal pump.

pump speed of 200 revolutions per minute is most efficient when pumping against a total head of about 18.5 feet and discharging 22,000 gallons per minute. For higher heads, the speed of the pump should be increased; and for lower heads, it should be decreased to secure the maximum efficiency.

When the pump is run at a constant speed of 275 revolutions per minute, the efficiency is 50 per cent when the head is 13.8 feet and the discharge is 39,500 gallons per minute, as indicated at point M. Continuing up the head-capacity curve, the maximum efficiency of 76 per cent is obtained at point P, where the head is 26 feet, the discharge 26,000 gallons per minute, and the

brake horsepower 225. At point *T*, the efficiency is again 50 per cent as shown at point *U* on the efficiency curve. Thus it is seen that to operate at the best efficiency the speed of this pump should be gradually increased from 200 revolutions per minute to 275 revolutions per minute as the head increases from 18.5 feet at *E* to 26 feet at *P*.

The value of characteristic curves lies in the fact that it is only through a study of them that the most efficient operating conditions can be determined.

Pumping Capacity Required.—The rate at which the pumps must remove water from a district depends upon the amount and distribution of the rainfall, the size of the district, the area and topography of hill lands contributing drainage water to the district, the nature of the soil, the storage capacity of the ditches and possible storage reservoirs, the amount of seepage under the levee, and the completeness of drainage desired.

The amount, distribution, and intensity of rainfall have been discussed in detail in Chapter II and will not be treated here. Since the rains which may cause damage are those which occur during the growing season, when evaporation and transpiration losses are high, and since the soil at this time rarely contains the maximum amount of capillary water, the run-off is comparatively small, except in cases of protracted storms of exceptional intensity.

The storage capacity of the ditches will vary with the design; but as pointed out in the chapter on drainage by open ditches, ditches for pumping plants are always made larger than ditches in other districts to provide additional storage, which makes smaller pumps possible and decreases the cost of the pumping, because the pumps will have to be stopped less often to allow the water to accumulate in sufficient quantities to supply the pumps. Generally, the storage capacity of the upper 5 feet of the ditches varies from 0.3 to 0.5 inch of water over the entire district. This capacity may be still further increased by a storage reservoir at the pumping plant.

If the levees are properly constructed, the amount of seepage water will not be sufficient to affect the choice of capacity of the pumps.

The pumping capacity is expressed in inches of depth of water over the entire drainage area, removed in 24 hours. The capacities of the pumping plants of districts in successful operation

along the Illinois River vary from 0.20 to 0.50 inch in 24 hours, with an average value of 0.36 inch.

In general it may be stated that for areas subject to from 35 to 40 inches of annual rainfall, a capacity of 0.25 inch is the minimum which should be used under the most favorable conditions of run-off and storage; a capacity of 0.50 inch for adverse conditions such as large quantities of hill water and small area of district; and about 0.38 inch for average conditions. The smaller the district, the larger is the needed capacity.

Where the precipitation is exceptionally large and the growing season long, pumping capacities as high as 1.50 inches may be necessary. The pumping capacities of seven districts in southern Louisiana are given in Table XLII.

TABLE XLII.—PUMPING CAPACITY OF LOUISIANA DISTRICTS[1]

Name of district	Area, acres	Pumping capacity, inches in 24 hours	Reservoir capacity, inches
Smithport Planting Company Tract	647	1.10	0.50
Willswood Plantation	2,600	1.40	0.42
Lafouche Drainage District No. 12 (Subdistrict No. 1)	835	1.70	0.40
Gueydan Drainage District (Subdistrict No. 1)	6,500	1.25	0.88
Des Allemands Polder No. 1	1,880	0.75	0.34
Jefferson Drainage District No. 3	5,000	1.25	0.60
Lafouche Drainage District No. 12 (Subdistrict No. 3)	2,250	1.15	0.50

[1] OKEY, C. W., "Run-off from the Drained Prairie Lands of Southern Louisiana," *Jour. Agr. Research*, vol. 11, no. 6.

The required pumping capacity should not be chosen arbitrarily, but only after a careful study of all the factors involved, since the first cost of the plant and its successful operation depend upon an intelligent determination of the rate at which the water must be pumped from the district to prevent injury to crops.

The run-off, reduced to cubic feet per second per acre (see Table XIII), multiplied by the number of acres contributing to the run-off gives the discharge in cubic feet per second which must be handled by the pumps.

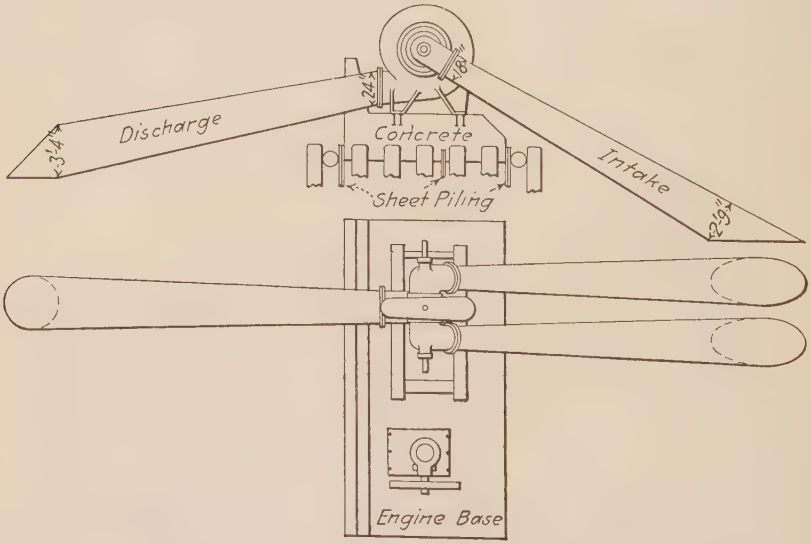
Choice of Pump Sizes.—The number and sizes of pumps necessary to deliver the computed discharge are determined from rating tables of the pump manufacturers, similar to Table XLI. Where the necessary capacity is greater than the capacity of about a 20-inch pump, it is better to use two small pumps than one large one. It is desirable to have two pumping units even in a small plant in order to have one pump in reserve in case of a breakdown. One of the pumps should be of such size that it can be operated continuously over long periods. If too large a pump is used, under ordinary conditions it will lower the water in the suction bay too rapidly, thus increasing the static head and possibly requiring the pump to be shut down until the water rises again. The frequent starting and stopping of pumps is especially undesirable where steam power is used on account of the cost of operation. Where two pumps are used, it is considered good practice to have the capacity of one of the pumps about twice that of the other. Where three or more pumps are required, they should be of the same size.

Elevation of Pump.—The elevation of the pump and the floor of the plant should be about 1 foot above the highest elevation to which the water in the suction bay can rise during periods of excessive rainfall with the pumps not operating. This means that the suction lift should be as low as is consistent with safety. Although the maximum theoretical suction lift is about 34 feet, the highest permissible suction lift is about 28 feet, because of losses due to friction in pipe and to velocity head. Most manufacturers will not guarantee a suction lift greater than 22.5 feet. It is not necessary that the pumps be placed higher than the highest water on the discharge side, although this is done in some plants, usually where the maximum static head is small.

Suction Pipes.—Suction pipes are generally riveted steel pipes, although reinforced concrete has been used recently in several installations.

The suction pipe should be strong, so as not to collapse under the greatest suction stress that will come upon it, perfectly airtight, smooth on the inside, and as straight as possible; and its lower end should be expanded gradually so that the entrance velocity of the water will be not more than 2 feet per second. In some installations the suction pipe is tapered the full length, but more often only the lower end is enlarged. If the surfaces are smooth and free from sharp angles, the reduction in cross-

section can be made comparatively sudden without appreciable loss of head. The area of the enlarged end should be four or five times that of the pump intake connection. The opening at the lower end may be either horizontal or vertical. If horizontal, the edge of the pipe should be about 1 foot below the lowest level of the water in the suction pit to prevent air from being sucked into the pipe. When the ends are cut vertically, whirlpools will form over the pipes and air will be admitted unless the water is at least 3 feet deep over the upper edge of the pipe. The suction pipe is usually placed on a uniform slope and rigidly



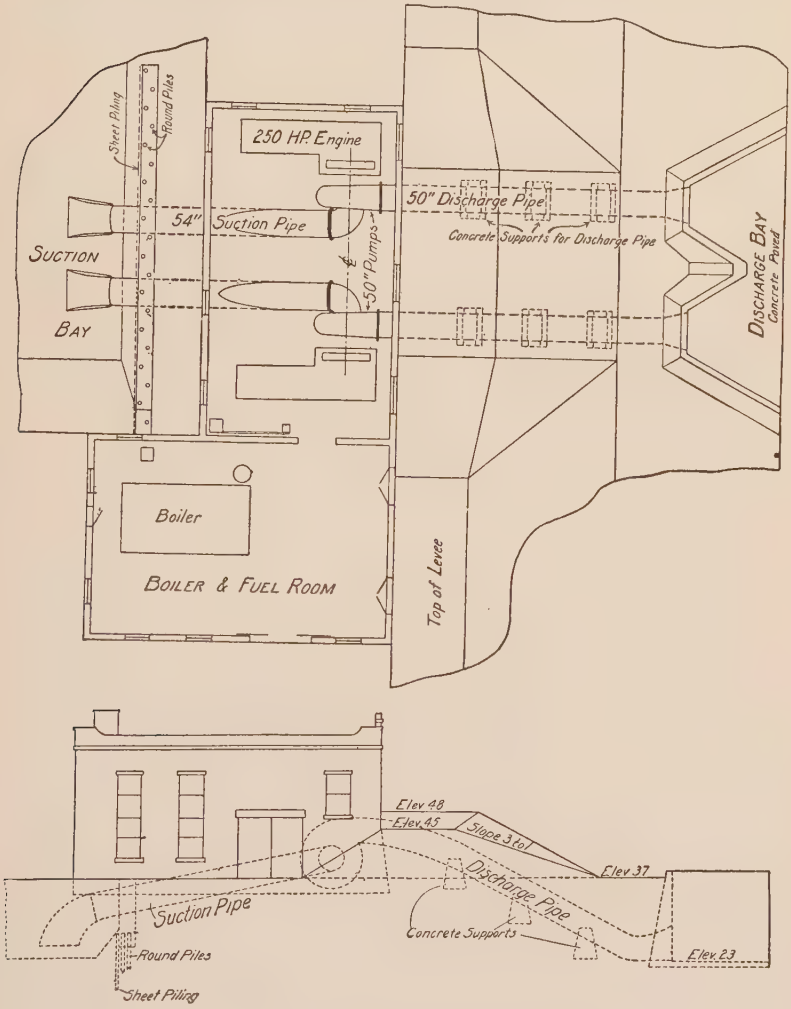
("*Bulletin* No. 71," U. S. D. A., page 45.)

FIG. 97.—Suction pipe with horizontal opening.

supported. Either the end of the pipe is cut horizontally, making an oblong opening, or the end of the pipe is fitted with an elbow of large radius as illustrated in Figs. 97 and 98. Metal pipes should be made of low-carbon steel, and after installation they should be coated with an asphalt paint. Figure 99 shows a plan and elevation of the pumping plant of the Muscatine-Louisa Levee and Drainage District, Iowa, in which the suction pipe is of reinforced concrete.

Discharge Pipes.—Riveted steel pipes are used for discharge pipes. They also should be strong, smooth, and straight. These pipes are gradually enlarged immediately after leaving the pump

to about twice the area of the pump discharge connection so that the velocity of discharge will not be more than 5 feet per second. This enlargement, however, must be more gradual than



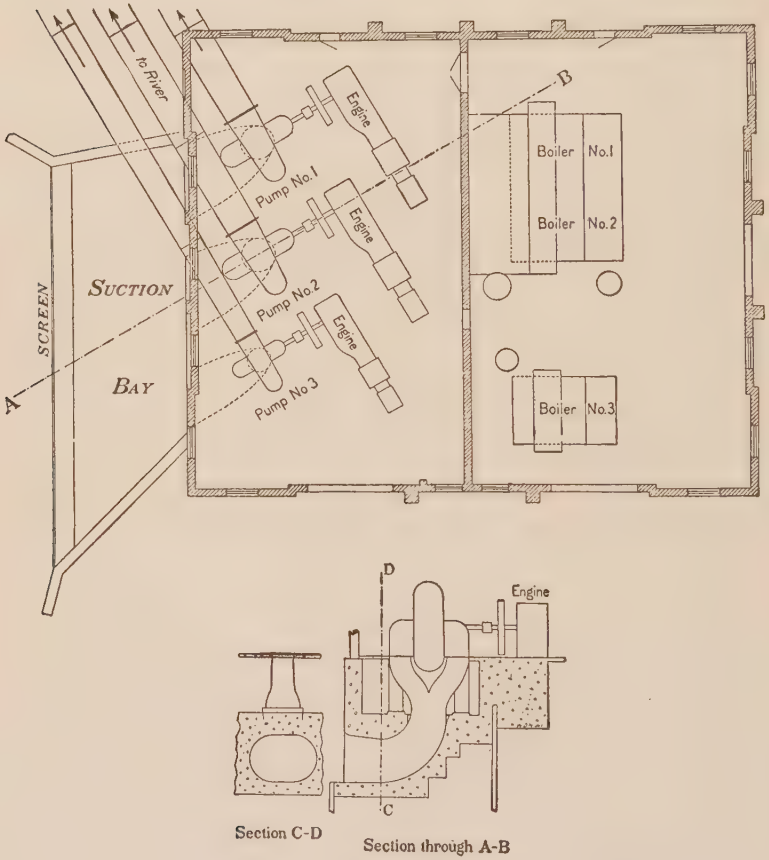
(*Bulletin* No. 304, U. S. Dept. of Agriculture.)

FIG. 98.—Suction pipe with elbow.

in the case of the suction pipe to prevent the loss of energy attendant upon a sudden enlargement of pipe cross-section. The outer end of the pipe may be either horizontal or vertical, similar to the suction pipe, and its elevation is such that it will always

be submerged, so that when the water in the discharge bay is lower than the pump, siphon action will prevent an increase in the static head due to the elevation of the pump.

There are two general arrangements of the discharge pipe. One is to have the pipe extend upward to the top of the levee and



(*Bulletin* No. 304, U. S. Dept. of Agriculture.)

FIG. 99.—Pumping plant of the Muscatine-Louisa draining district.

then down to the discharge bay, so that the bottom of the pipe at its highest point is above high water, thus preventing the water outside from flowing back through the pump when it is not operating. The other arrangement is to have the pipe pass through the levee either on a level or on a downward slope. In the first arrangement an opening is necessary in the pipe at

its highest point for priming the pump in starting and for admitting air to break the water seal and prevent water flowing into the district when the pump is stopped. Provision must also be made for removing the air which will collect in the top of the pipe during pumping and interfere with the flow of water through the pipe. In the second arrangement, a flap valve will be required on the end of the pipe if the pipe is small; if large, a slow-closing type of gate valve close to the pump should be used. Also, concrete collars or cut-off walls should be provided at intervals along the pipe to prevent seepage of water through the levee. Because of danger to the levee, the Mississippi River Commission has ruled that on all new installations no pipe shall pass through the levee.

Suction Pit.—The suction pipes should be submerged in a suction pit adjoining the pumping plant. The sides of the pit should be built of reinforced concrete, supported on piles, and with sheet piling driven under the walls to prevent seepage. To keep out floating trash, a screen should be placed across the entrance to the pit. The opening in the screen should be two or three times the combined area of the openings in the suction pipes. The bottom of the pit should be about one and one-half times the diameter of the opening in the suction pipe below the edge of a horizontally-cut suction pipe. For a vertical opening in the suction pipe, the depth of the pit should be about 3 feet greater than the diameter of the opening. If the material in the bottom of the pit is quicksand, the bottom should be concreted and tied down by piling so as to resist the upward pressure of the water in the soil when the water in the pit is low and the water on the outside of the levee is high.

Discharge Bay.—The discharge pipes should empty into a discharge bay designed to prevent erosion and undercutting of the ends of the pipe. This is accomplished by a concrete head wall with wing walls; and if the pipes are not below the low-water elevation of the river, the bottom of the discharge bay should be heavily riprapped or paved with concrete. This outlet structure should be of ample dimensions and constructed with special care.

Sluiceways.—In most districts, the water level of the river in the summer is low enough to permit the discharge of water from the district by gravity; and in this case, a sluiceway should be built under the levee near the pumping plant. A sluiceway consists of one or more lines of pipes, protected at the ends by

suitable head walls, and provided with valves for opening and closing. The pipes should be surrounded by cut-off walls at intervals to prevent seepage of river water along the outside surface of the pipes.

A sluiceway should be designed for a run-off modulus of 0.25 inch in 24 hours, and the capacity of the pipes should be such that the discharge velocity will be not more than 5 feet per second, in order to reduce friction losses to a pratical minimum.

Foundations.—Special care must be taken in designing and constructing the foundations for the pump and engines. The foundation, if constructed properly, is the most permanent part of the pumping plant. The foundation should be supported on timber piles of such a length that their tips will penetrate into a hard stratum. The necessary length of piles is determined by borings. The entire superimposed load should be assumed to be carried by the piles. The number and spacing of piles will depend upon the loads to be carried. Ordinarily, a spacing of about 3.5 feet center to center is good practice. The tops of the piles should extend several feet into the concrete block which rests upon them, and the bottom of the concrete base should be below the permanent water level to protect the piling from decay. It is also advisable to drive one or two rows of sheet piling around the foundation. The foundation proper should be a massive block of concrete, well reinforced with steel, of such size and dimensions as to support both the pumps and the engines, especially if the engines and pumps are directly connected. When steam is used for power, a foundation for the boilers is also necessary. The latter is generally separate from the pump and engine foundation.

The building which houses the machinery should also have concrete foundation walls, supported upon piles.

Operation of Centrifugal Pumps.—In starting a centrifugal pump, the first thing to do is to close the valve in the discharge pipe to prevent the water from siphoning backward through the pump after it has been primed and before the pump is started, thus throwing an overload on the motor or engine in starting, since more power is required at normal delivery than at shut-off. The next step is to prime the pump by exhausting the air in the pump and pipes. This is done either by an air pump or a steam ejector. The pump can then be started and brought up to speed, after which the discharge valve is opened slowly, so as not to

throw a sudden load on the engine. A centrifugal pump should never be run empty, since certain parts of the pump depend upon water for their lubrication.

Screw Pumps.—The screw pump consists of a horizontal shaft with many blades attached along its axis, which rotates in

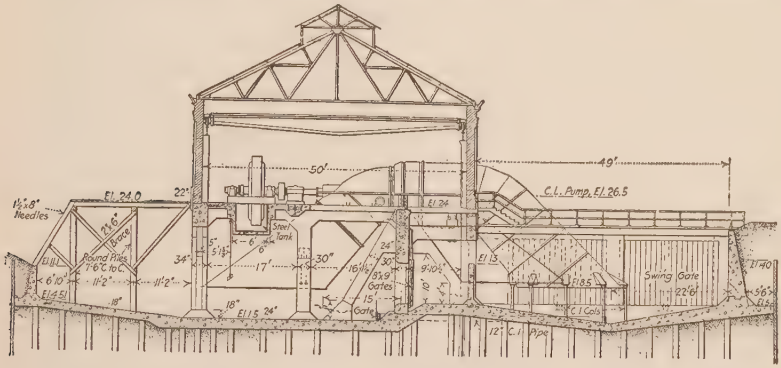


FIG. 100.—Elevation of pumping station at New Orleans, showing arrangement of 12-foot screw pump.

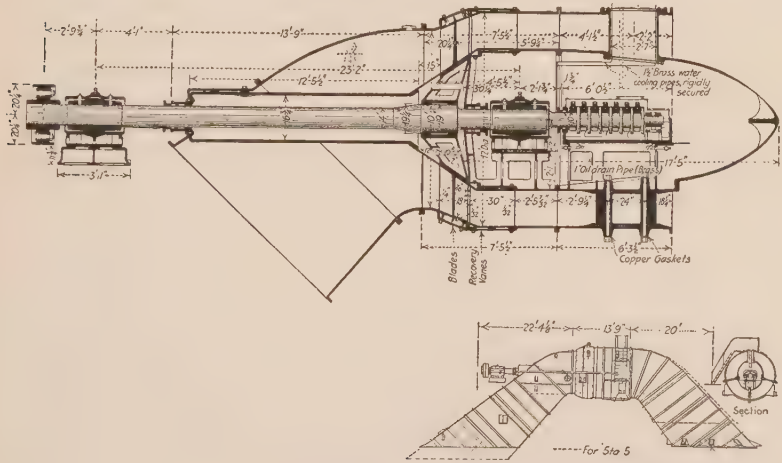


FIG. 101.—Plan, elevation, and section of 12-foot screw pump for New Orleans drainage stations.

a more or less cylindrical casing. The blades are very similar to those on a boat propeller. In the smaller pumps, the shaft extends through the casing and both bearings are outside; in the larger sizes, one shaft bearing is inside the casing.

The revolving blades give a spiral motion to the water as it is forced horizontally through the pump. Diffusion vanes on the

discharge end change this spiral motion into rectilinear motion as the water enters the discharge pipe.

The screw pump is particularly adapted to low lift conditions, such as exist along the coast. It is designed for much higher velocities than the centrifugal pump, and its characteristics differ in that the horsepower decreases as the head decreases. Its efficiency is higher through a greater range of heads than that of the centrifugal pump, especially in the larger sizes.

Two 78-inch screw pumps are used by one drainage district in Louisiana; and twelve 12-foot screw pumps have been installed by the city of New Orleans for removing surface water. The elevation of one of these pumps is shown in Fig. 100, and a sectional view of the pump itself is shown in Fig. 101.

PRIME MOVERS

Steam engines, electric motors, and internal combustion engines are all used as sources of power for pumping plants. The choice of prime mover in any specific case will depend upon the amount of power which must be developed, the initial cost, the availability and cost of fuel or electric power, the conditions under which the pumps must be operated, and the availability of skilled labor. Each kind of prime mover will be discussed separately and the advantages and disadvantages of each and the conditions under which each is most suitable will be given.

Steam Engines

The steam engine has been used more in drainage pumping than any other kind of prime mover. Its advantages are: (1) reliability; (2) excellent regulation of speed for pumping against varying heads; (3) large overload capacity; (4) comparatively low first cost; (5) economy in operation where coal or fuel oil can be obtained cheaply and the amount of pumping is large; and (6) the availability of enginemen for operating plant. Its disadvantages are: (1) delay in starting pumps due to getting up steam in the boilers; (2) loss of power when pumping intermittently; (3) large depreciation, especially of boilers, when plant is operated for a few months only during the year; and (4) large floor space required for engines and boilers.

Three types of steam engines are ordinarily used for drainage pumping, the plain slide valve, the Corliss, and the automatic high-speed.

Plain Slide-valve Engine.—This type of engine is simple in construction, low in first cost, and easy to operate, and does not get out of order easily; but, since the speed is regulated by throttling, or reducing the pressure of the steam supply, it is uneconomical in the use of steam, requiring from 35 to 60 pounds of steam per horsepower-hour. For this reason it is usually made in small sizes and is designed to run at low or medium speeds, up to about 200 revolutions per minute. A number of the smaller pumping plants in Louisiana are operated by this type of engine, varying in size from 10-inch diameter of piston and 14-inch length of stroke to 16-inch piston and 24-inch stroke.

Corliss Engine.—This is the most efficient type of steam engine, rarely using over 25 pounds of steam per horsepower-hour, and in the larger sizes even less than this amount. It is made in sizes from 100 to 1,200 horsepower. The Corliss engine is essentially a slow-speed engine, because of its complicated valve mechanism, and is rarely designed to run at speeds higher than 125 revolutions per minute. The cylinder of this engine is long in proportion to the diameter to maintain a proper piston speed at the low engine speed. The speed is regulated by changing the point of cut-off to suit the load, thus controlling the volume of steam admitted to the cylinder without changing the admission pressure of the steam, which is constant during the entire period of admission. The Corliss is a very smooth-running type of engine and its speed can be closely regulated.

Automatic High-speed Engine.—This type of steam engine is used to some extent in drainage work and is growing in favor for the larger plants where more than 200 horsepower and high speeds are required. Because of the high engine speed, the length of cylinders is short in proportion to the diameter. The speed is controlled by changing the point of cut-off, as in the Corliss engine, thus using the steam at boiler pressure; but the pressure gradually decreases up to the point of cut-off. This engine is made for speeds up to about 350 revolutions per minute and in sizes up to about 600 horsepower; but where more than 300 horsepower are required of a single engine, it may be desirable to use the "uniflow engine" with superheat, which is more economical of steam than any other type. The automatic

high-speed engine uses from 30 to 40 pounds of steam per horsepower-hour. Also, this type of engine requires more experienced labor to operate than the other types.

Condensing and Non-condensing Engines.—Steam engines are also classified as condensing and non-condencing, depending on the method of handling the exhaust steam. In a non-condensing engine the steam is exhausted from the cylinder into the atmosphere, and consequently the exhaust stroke of the piston is made against atmospheric pressure; while in a condensing engine the steam from the cylinder is exhausted into a closed vessel and condensed into water, thus creating a partial vacuum and reducing the pressure against which the exhaust stroke is made. A condensing engine is more efficient than a non-condensing one and uses about 20 per cent less steam to develop the same amount of power, because the steam is expanded a greater number of times in the former engine and hence the volume of steam admitted to the cylinder can be made smaller by shortening the admission stroke.

There are two general types of condensers used, the jet condenser and the surface condenser. In the jet condenser, the steam is condensed by passing it through a spray of water in the condensing chamber, and the condensate and condensing water mix and collect in the bottom of the chamber where it is removed by a pump or by siphon action. Where the condensing water is suitable for feeding the boilers, this type of condenser is preferable.

In the surface condenser, the condensing water and the steam do not come into direct contact, but the steam is condensed by contact with numerous water-filled tubes or else the steam passes through water-jacketed tubes. The condensate in this case can always be used in the boilers.

Engines used in pumping plants should be condensing on account of the greater efficiency thus obtained.

Simple and Compound Engines.—Steam engines may be further classified as simple, compound, and triple-expansion, depending upon the number of cylinders in which the steam is expanded.

To use steam efficiently in a simple engine, the point of cut-off must come early so as to allow a large ratio of expansion of the steam; but, on the other hand, a large number of expansions of steam in a single cylinder increases the amount of condensation of steam inside the cylinder because of the large range of tem-

perature to which the walls of the cylinder are subjected, due to the large expansion of the steam. Hence, a simple engine is not well adapted to the high steam pressures nor to the short-steam admission stroke which are necessary for steam economy.

The chief advantage of a compound engine over a simple engine is that in it the losses from condensation in the cylinders are greatly reduced, since the total steam expansion takes place in two cylinders instead of in one, thereby decreasing the range of temperature of the cylinder walls in each cylinder. Compounding an engine does not increase its power, but simply reduces the cost of operation. Therefore, for large plants using high steam pressures, the compound engine is better adapted to the economical use of steam.

Boilers.—Both fire- and water-tube boilers are used in drainage pumping plants. When properly designed and operated, both types have about the same efficiency. The fire-tube boiler, however, is better adapted to small plants up to 200 horsepower, requiring steam pressures up to about 125 pounds; and for such plants the first cost is less than that of a water-tube boiler of the same capacity. The water-tube boiler is more suitable for large plants requiring high steam pressures.

The fire-tube boiler consists of a cylindrical shell closed at the ends by two flat heads, and of numerous fire-tubes extending the entire length of the lower two-thirds of the shell. The shell is filled with water to above the level of the fire-tubes, the remaining space being left for the collection of steam. The grate of this boiler is at the front, and the hot gases pass under the shell to a combustion chamber at the rear of the shell, thence through the fire-tubes inside the shell to the front, and from there to the breeching and the chimney.

Fire-tube boilers are of three kinds, the horizontal return tubular, the Scotch marine, and the locomotive. The first of these is inclosed in a brick setting and is externally fired; whereas, the last two are not inclosed and are fired internally—the grate, the boiler proper, and the combustion chamber being constructed as a unit. The marine and locomotive types are suitable for very small plants only. The return-tubular boiler is the type most commonly used. It is simple, inexpensive, and has a large overload capacity. The fire-tubes are generally 3 or 4 inches in diameter and from 14 to 18 feet in length.

The water-tube boiler consists of numerous tubes filled with water and one or more drums above and connected to the tubes for disengaging the steam from the water. There are many makes of water-tube boilers, varying in the position of the tubes and in the path taken by the hot gases from the grate to the stack; but in most of them the hot gases pass across or along the tubes three times on their way to the stack. These boilers are inclosed in a brick setting. Water-tube boilers, as a rule, generate steam faster than fire-tube boilers and are safer, but are more expensive.

Boiler Horsepower.—Boiler horsepower is an arbitrary unit used in rating the capacity of steam boilers. The definition recommended by the American Society of Mechanical Engineers is that 1 boiler horsepower is the evaporation of 34.5 pounds of water per hour from feed water at 212°F. into dry steam at the same temperature. The amount of heat required to develop 1 boiler horsepower is 33,455 British thermal units per hour.

There is no definite relation between boiler horsepower and engine horsepower. A steam engine will develop 1 indicated horsepower upon a water consumption of from 10 to 60 pounds per hour, depending upon whether the engine is simple or compound, condensing or non-condensing, or uses saturated or superheated steam. Thus 1 boiler horsepower will correspond to from 3 to 0.5 engine horsepower. In designing large pumping plants a ratio of 1.5 to 1 is considered good practice. For small plants, however, about 1 boiler horsepower should be provided for each indicated horsepower of the engine, since it is extremely important that the boiler capacity be sufficient to supply safely all the steam the engines will need when pumping against a maximum head.

A boiler should develop its rated capacity under ordinary conditions of operation; but by using full draft and more fuel, it should develop at least one-third more than its rated capacity.

The capacity of a boiler varies with its heating surface. From 10 to 12 square feet of surface in contact with the hot gases is required to develop 1 boiler horsepower. It is generally assumed that 1 pound of coal will evaporate about 10 pounds of water, and that 1 pound of crude oil burned under a boiler will evaporate from 12 to 18 pounds of water.

The efficiency of a boiler will depend upon the design and construction of the boiler and the furnace, the quality of the fuel,

and the manner of operation. Under average conditions a boiler should have an efficiency of about 70 per cent.

Boiler Auxiliaries.—The more important boiler auxiliaries are feed-water heater, feed pumps, and chimney or stack. These auxiliaries should be ample in capacity so as not to limit the capacity of the boilers and engines under unusual conditions.

The water for the boilers is generally heated by exhaust steam. Where a surface condenser is used in condensing the exhaust steam from the cylinders, the water from the condensed steam is used over and over again in the boiler. Where a jet condenser is used, the mixed condensing water and the condensate can be used in the boiler if the quality of the condensing water is suitable.

Generally, a direct-acting steam pump is used for forcing the feed water into the boilers. If the temperature of the feed water is not over 150°F., a steam injector may be used.

Chimneys are necessary not only for carrying off the waste gases but also for creating a draft under the boilers. The intensity of chimney draft is generally from $\frac{1}{2}$ to $\frac{3}{4}$ inches of water. The area and height of a chimney will depend on the horsepower to be developed by the boilers. The relation between these three factors is expressed by the formula,

$$\text{Boiler horsepower} = 3.33(A - 0.6\sqrt{A})\sqrt{H} \qquad (36)$$

in which A is the cross-sectional area of the chimney in square feet, and H is the height of the chimney above the grate in feet.

Engine and Boiler Units.—In large plants the pumps, engines, and boilers should be divided into two or more equal units to avoid the loss incident to operating large units when only a small amount of pumping is to be done.

Pump-engine Connection.—Centrifugal pumps may be either directly connected to the steam engine or driven by a belt or a rope. Pumps smaller than about 24 inches cannot be directly connected to the engine because the speed at which such pumps must be driven is generally higher than the engine speed. For large pumps, a direct connection should be used. This may be done in several ways: (1) The pump shaft may be made solid with the engine shaft; (2) the two shafts may be connected by flexible couplings, which is the usual method; and (3) the shafts may be connected through friction-clutch couplings.

Electric Motors

Electric motors are well adapted to the operation of centrifugal pumps where adequate, reliable transmission lines are readily accessible and electric current can be purchased at a reasonable rate.

The advantages of electric motors are: (1) low first cost; (2) convenience of operation; (3) low depreciation; (4) readiness for instant operation; (5) small floor space required; (6) no storage space for fuel; and (7) no losses when not in use.

The disadvantages are: (1) high cost of operation; and (2) constant speed when alternating current is used—which means that the pump can be operated efficiently at only one head.

The three-phase induction motor is the most suitable for the operation of centrifugal pumps since it has a high efficiency over a wide range of heads. There are two types of induction motors, the squirrel-cage and the wound-rotor, both of which are used in drainage pumping.

Squirrel-cage Motor.—The distinguishing feature of the squirrel-cage motor is the method of winding the rotor or armature. The winding consists of copper bars fitted into slots in the rotor core and connected at their ends by conducting rings. This method of winding gives a very low and constant resistance in the rotor, and causes the motor to operate at substantially constant speed. The running characteristics of the squirrel-cage motor are excellent because of its low rotor resistance; but, on the other hand, its starting characteristics are poor, since it requires a large current at low power factor on starting, and even then develops but little torque. For this reason, this type of motor is generally used in small sizes only, and where a large starting current is not objectionable, and where it need not be started under any considerable load. It is simple in construction, however, is less expensive than the wound-rotor motor, and requires but little attention in operation.

Wound-rotor Motor.—The rotor of this type of motor is wound two- or three-phase—usually the latter—similar to the winding of the stator. The ends of the three-phase winding are connected to three slip rings on the rotor shaft. By means of brushes bearing on each slip ring, external resistances are connected to the rotor through a controller. This gives the motor a high starting torque with a moderate amount of current from the line,

and this is the principal advantage of this type of motor. When the motor is up to speed, the external resistance is cut out and the motor operates at a constant speed similar to the squirrel-cage motor; but it has a lower efficiency because the rotor has a greater slip due to the greater resistance in the rotor windings. By introducing external resistance in the rotor circuit, the slip is increased and the speed reduced, hence this motor can be changed to a variable speed motor; but the lowering of the speed lowers the efficiency of the motor and gives poorer speed regulation. The smaller the resistance in the rotor, the less is the slip (difference between synchronous speed and rotor speed), and the higher the efficiency of the motor. Because of the greater slip of the wound-rotor motor at running speed, its efficiency is less than that of the squirrel-cage motor. The maximum speed of an induction motor is always less than its synchronous speed.

This motor can run indefinitely at 1¼ nominal load; but if overloaded to two or three times its nominal load, it will stop. In the larger sizes, the efficiencies are good for a wide range of loads, ranging from about 87 per cent at half load to 89 per cent at full load, and about 88 per cent at 1¼ load. Below ½ load, the efficiency drops rapidly.

In general it may be stated that the three-phase, wound-rotor, induction motor, constructed so that all resistance external to the rotor can be cut out, is the most suitable type of motor for use with centrifugal pumps of the larger sizes.

Current is generally supplied at 1,100 volts from the transmission line and stepped down outside the pumping plant to 440 volts.

Motor-pump Connection.—Motors are connected to pumps in three ways: (1) by direct connection; (2) by belt and pulley; and (3) by chain and sprockets.

In the first of these, there is no way to change the pump speed when pumping against variable heads; hence, this is the least desirable of the three methods.

In the belt connection, several pulleys of different sizes are keyed to the pump shaft, and the belt changed from one pulley to another to vary the speed of the pump to conform to the head pumped against. The chief objection to this method is the loss of power due to slipping of the belt.

The silent chain drive is the method most used at the present time. Sprocket wheels of two or more sizes are fitted to the

motor shaft to provide for a change of speed. In a recent installation, two sprockets of different sizes are mounted permanently on the motor shaft, while the sprocket on the pump shaft is arranged to slide horizontally on the pump shaft, so that it can carry the chain for either of the driving sprockets.

Internal Combustion Engines

Some of the more recent pumping plant installations use internal combustion engines as a source of power. Internal combustion engines adapted to this purpose burn heavy liquid fuels of high boiling points directly in the engine cylinder; therefore, engines of this kind are commonly called "oil engines" to distinguish them from the more common form of internal combustion engine which uses gasoline as a fuel.

Oil engines differ chiefly from gas engines in that no carburetor is used for vaporizing the fuel and mixing it with air before it is admitted to the engine cylinder, and that no electric spark is used to ignite the mixture in the cylinder. In the oil engine, the liquid fuel is injected under high pressure directly into the cylinder at the end of the compression stroke when the cylinder contains only highly compressed air, and is there vaporized and the mixture ignited automatically either by the highly heated air or by contact with a highly heated metal surface.

Advantages and Disadvantages.—The advantages of the oil engine as a prime mover are: (1) its unsurpassed fuel economy in all-size units; (2) its readiness for instant operation; (3) the fact that it is a self-contained prime mover, its only auxiliary being an air compressor; (4) the small space required for the storage of fuel; (5) no losses in getting started or in operating, as is the case with the steam engine; (6) good speed regulation throughout a large range; (7) high efficiency at low speeds; and (8) reliability when properly designed and intelligently operated. Its disadvantages are: (1) high first cost; and (2) the fact that it cannot be run satisfactorily above its nominal rating.

Types of Oil Engines.—There are three general types of oil engines: (1) the Diesel engine in which the combustion is at constant pressure; (2) the explosion oil engine, more commonly known as the semi-Diesel, in which the fuel is burned at constant volume; and (3) the Sabathé engine which combines the features of the other two types and burns the fuel at both constant pres-

sure and constant volume. Each of these types is made to work on either a two- or a four-stroke cycle. The first and second types of oil engines are best adapted to drainage pumping plants and will be discussed more fully.

The Diesel Engine.—In the Diesel engine, air only at atmospheric pressure is admitted to the cylinder during the suction stroke. On the compression stroke this air is compressed to a pressure of from 450 to 500 pounds per square inch and its temperature thereby raised to about 1000°F. At the end of the compression stroke, fuel, proportional in amount to the load on the engine, is injected into the cylinder by air at a pressure of from 700 to 900 pounds per square inch and is thereby atomized. The high temperature of the compressed air in the cylinder ignites the atomized fuel which burns—rather than explodes—at a constant pressure during the entire time that the fuel is being injected into the cylinder. The expanding gases give the impulse stroke to the piston. The fuel is pumped into the injection chamber at a pressure of from 800 to 1,100 pounds per square inch during the suction stroke of the engine when the injector valve is closed. The engine governor controls the amount of fuel pumped into the injection chamber, and in this way the amount of fuel is automatically proportioned to the load on the engine.

In the four-stroke engine only one power impulse is given to the piston in two revolutions; whereas, in the two-stroke cycle there is an impulse stroke for each revolution. In the former engine, the four events of suction, compression, expansion, and exhaust each take place during one-half a revolution; in the latter, the exhaust or scavenging stroke and the suction stroke are combined and take place during about three-sixteenths of a revolution, the compression stroke occupies about three-eights of a revolution, and the expansion stroke about seven-sixteenths of a revolution. The two-stroke engine requires a scavenging pump, which forces air at a pressure of from 3 to 7 pounds per square inch into the cylinder at the end of the expansion stroke, for driving out the products of combustion through exhaust ports around the cylinder walls uncovered by the piston, and for filling the cylinder with fresh air.

Since a two-stroke engine receives a power impulse for every revolution, it would seem to be superior to the four-stroke engine which receives a power impulse every two revolutions; but, on

the other hand, the two-stroke engine requires a large air pump for scavenging the cylinder of the burned gases. Also, these burned gases are not so completely removed in the two-stroke as in the four-stroke engine, which may result in loss of power. Further, two-stroke engines should not be designed for as high mean effective pressure nor as high piston speeds as the four-stroke engine on account of the more rapid heating of the former engine.

In general it may be stated that the two-stroke engine is preferable for small plants, while the four-stroke engine is better adapted to large plants.

The Diesel engine requires the best grade of materials in its construction and the highest quality of workmanship, which make it a very high-priced engine. They are manufactured in sizes from 75 horsepower up, are designed to run from 150 to 350 revolutions per minute, and cost about $90 per horsepower. Because of the high initial temperature due to high compression, the Diesel engine can use heavier and cheaper fuels than other oil engines. It will develop 1 brake horsepower-hour on about 0.5 pint of fuel oil.

Explosion Oil Engine.—During the suction stroke air is admitted to the cylinder at atmospheric pressure. On the compression stroke the air in the cylinder is compressed to a pressure of from 150 to 220 pounds per square inch. Just before the piston reaches the end of the compression stroke, fuel is sprayed by a fuel pump into a circular, unjacketed chamber, or ball, connected to the cylinder. This ball is heated to a red heat, before starting the engine, by means of a torch, and after the engine is running, the repeated explosions keep it hot. The fuel coming in contact with the heated surface of the ball is vaporized and burned with explosion-like rapidity. The expanding gases give a power impulse to the piston.

Except for the means of atomizing and igniting the charge, the four-stroke explosion engine operates in the same manner as the Diesel engine.

The explosion oil engine having a two-stroke cycle has a closed crank case with large disk valves on one side. These valves open inwardly and serve as air intakes. The crank case is connected to the lower end of the cylinder by a large air passage. On the expansion stroke, the air in the crank case is compressed

to a pressure of from 1 to 7 pounds per square inch. Shortly before the piston reaches lower dead center, the piston head uncovers the port of the air passage from the crank case and also the exhaust port. The compressed air from the crank case forces the burned gases from the cylinder and admits fresh air. As the piston travels upward, it covers the ports, compresses the air in the cylinder, and creates a partial vacuum in the crank case which causes the outside air to enter through the air intakes, thus completing the two-stroke cycle.

Explosion oil engines cost only about two-thirds as much as Diesel engines, but they are not as economical of fuel and lubricating oil as the latter engine, especially when running at less than full load, because of the lower initial compression in the explosion oil engine. The lower compression is necessary, however, in an engine in which the fuel is burned at constant volume, otherwise the maximum pressure and temperature in the engine would exceed safe limits. In the Diesel engine the fuel is burned at constant pressure and hence a higher compression can be used with safety. The lower first cost of the explosion oil engine more than offsets the higher fuel costs when small plants are considered; and for such plants, this engine with a two-stroke cycle is more suitable than the more expensive Diesel engine. The speed of this engine is relatively high, varying from 200 to 260 revolutions per minute for sizes from 50 to 150 horsepower. An explosion oil engine will develop 1 brake horsepower-hour on about 0.7 pint of fuel oil.

Cylinder Units.—Although single-cylinder oil engines of large power are manufactured, it is preferable to use two or more cylinders where more than 100 horsepower are required or when a four-stroke engine is used, since multicylinder units give a greater uniformity of motion, decrease the weight of the reciprocating parts of the engine, and require smaller dimensions of the individual units, thus making a lighter and more compact engine. Two-cylinder, four-cycle engines are built in sizes from 50 to 300 horsepower; and three- and four-cylinder engines from 100 horsepower up. In two-cycle engines, one-half the number of cylinders will produce the same effect.

Fuel Oil.—Gas oils, or oils which are distilled at relatively high temperature from crude oils, and light petroleums, are the only fuels suitable for explosion oil engines; whereas, Diesel engines can use such heavy liquid fuels as petroleum residues, coal-tar

oils, and coal tars. The coal-tar oils, however, can be used successfully only in large engines, operated continuously at about full load, and even then gas oil must be used in starting. In the United States, petroleum fuels are commonly used in operating Diesel engines.

PUMPING PLANT DESIGN

Data Required.—In designing a pumping plant it is necessary to have the following data: (1) the maximum rate at which the water must be pumped; (2) the maximum, average, and minimum static head; (3) the availability and cost of fuel and electric power; and (4) foundation data.

Static Head.—The static head is the distance in feet through which the pump must lift the water. It is the difference in the surface elevations of the water in the suction bay and in the discharge bay. The maximum static head for drainage districts is generally about 20 feet, and the average static head from 5 to 10 feet.

Dynamic Head.—The dynamic head, or total head, is the sum of the static head, the head lost at entrance to the suction pipe, the head lost at end of discharge pipe, and the friction head in the two pipes.

The amount of head lost at entrance to the suction pipe depends chiefly on the velocity of the water passing through the pipe. If the suction pipe is of uniform diameter, the loss of head at entrance is about $0.62 \frac{V^2}{2g}$; and for a velocity of 10 feet per second in the suction pipe, amounts to about 1.0 foot. If, however, the end of the suction pipe is enlarged, say to twice the diameter of the suction-pipe connection on the pump, the entrance velocity will be only one-fourth the velocity through the pump, and the head lost at entrance will be only one-sixteenth that for a suction pipe of uniform size. For a velocity through the pump of 10 feet per second, the loss at entrance is only about 0.06 foot.

The velocity of the water at discharge represents kinetic energy created by the pump, which is wasted. This lost head at discharge is the velocity head due to the discharge velocity, and represents the head necessary to start the water in the pipe in motion and to bring its motion up to normal velocity. The lost head at discharge is, therefore, represented by the expression

$\frac{V^2}{2g}$, in which V is the velocity at discharge. Generally, the discharge pipe is enlarged so that the discharge velocity is about 5 feet per second; and for this velocity, the loss of head at discharge is approximately 0.4 foot.

The loss of head due to friction depends upon the diameter and length of suction and discharge pipes, and upon the velocity. These losses for each 100 feet in length of clean, straight iron pipe of various diameters for a velocity of 8 feet per second are as follows:

Diam. of pipe (inches).....	15	18	21	24	27	30	36	42	48	54	60
Friction Head (feet).....	1.63	1.36	1.16	1.02	0.90	0.81	0.68	0.58	0.51	0.45	0.40

If a sharp bend or elbow is introduced, there will be an additional friction loss of approximately 1 foot.

Number and Sizes of Pumps.—The total capacity of the pumps is computed by multiplying the number of acres served by the pumps by the chosen run-off modulus, expressed in cubic feet per second per acre, or gallons per minute per acre. This total capacity may be handled by one, two, or three pumps, rarely more. The number of pumps in any specific case will be determined by the total capacity and the capacity needed for average rainfall conditions. For example, experience has shown that for the Illinois River districts the total pumping capacity should be sufficient to remove 0.3 inch in depth over the drainage area in 24 hours; but, on the other hand, pumping records show that for a large part of the year only about one-third of this capacity is necessary. For this reason, the smallest pump should have a capacity of about one-third the total capacity. Table XLIII shows approximately the number and sizes of pumps required in districts of from 1,000 to 20,000 acres for removing 0.3 inch in 24 hours, the capacity of the pumps being based on a velocity of 10 feet per second through the pump. The Table shows that for areas up to 1,000 acres, only one pump is recommended; between 1,000 and 8,000 acres, two pumps of different size should be used, the capacity of one being about twice that of the other; between 8,000 and 10,000 acres, either two pumps of unequal size may be used or three pumps of the same size; and for

areas over 10,000 acres, three or more pumps of the same size should be used.

TABLE XLIII.—NUMBER AND SIZES OF PUMPS FOR DISTRICTS OF VARIOUS SIZES

Size of district in acres	Capacity of pumps for 0.3-inch drainage modulus		Number and sizes of pumps recommended
	Cubic feet per second	Gallons per minute	
1,000	12.6	5,650	1—15 in.
2,000	25.2	11,300	2—15 in.
3,000	37.8	16,950	1—15 in. and 1—22 in.
4,000	50.4	22,600	1—18 in. and 1—24 in.
5,000	63.0	28,300	1—20 in. and 1—28 in.
6,000	75.6	33,950	1—22 in. and 1—30 in.
7,000	88.2	39,600	1—24 in. and 1—32 in.
8,000	100.8	45,250	1—24 in. and 1—36 in., or 3—24 in.
9,000	113.4	50,900	1—26 in. and 1—39 in., or 3—26 in.
10,000	126.0	56,550	1—28 in. and 1—39 in., or 3—28 in.
12,000	151.2	67,850	3—30 in.
14,000	176.4	79,150	3—32 in.
16,000	201.6	90,500	3—36 in.
18,000	226.8	101,800	3—36 in., or 4—32 in.
20,000	252.0	113,100	3—39 in., or 4—36 in.

Water Horsepower.—The water or theoretical horsepower developed by a pump at maximum head and rate of discharge may be computed from equations 37 or 38.

$$\text{Water horsepower} = \frac{62.5hQ}{550} = 0.1136hQ \qquad (37)$$

in which h is the dynamic head in feet, and Q is the maximum discharge in cubic feet per second.

If the discharge, Q, is in gallons per minute, the equation becomes,

$$\text{Water horsepower} = 0.0002525hQ \qquad (38)$$

Brake Horsepower.—The brake horsepower is the amount of power which the engine must deliver to the pump shaft in order

that the pump may develop a given horsepower. If the efficiency of the pump at a given discharge and head is 70 per cent, the brake horsepower will be 1.43 times the water horsepower; and for pump efficiencies of 60 and 50 per cent, the brake horsepower will be 1.67 and 2, respectively, times the water horsepower. The brake horsepower is always computed from the maximum water horsepower by dividing the latter by the efficiency developed by the pump when it is delivering its rated capacity at maximum head.

Indicated Horsepower.—The indicated horsepower is the power developed within the cylinder of the engine. As this power is delivered through the piston rod, cross-head, connecting rod, crank shaft, and main shaft to the flywheel, a part of it is consumed in overcoming friction, and hence the brake horsepower delivered to the pump shaft is only from 85 to 95 per cent of the indicated horsepower; or the indicated horsepower is about 10 per cent greater than the brake horsepower.

Illustrative Example.—What pumping equipment and power will be required to remove 0.3 inch of water from a district of 12,000 acres, if the maximum static head is 20 feet and the average static head is 10 feet? What kind of prime mover should be used?

To remove 0.3 inch in depth of water over an area of 12,000 acres in 24 hours, the water must be pumped at a rate of approximately 151 cubic feet per second or 67,850 gallons per minute. This can be done either by one 54-inch pump, two 39-inch pumps, one 32-inch pump and one 42-inch pump, or by three 30-inch pumps. Since it is desirable to have at least two pumps, and since the smallest pump should have about one-third of the total capacity, the 54-inch pump and the two 39-inch pumps are eliminated from consideration. Further, in the larger-size pumps there is an advantage in having pumping units of uniform size. Also, a 30-inch pump comes nearer having one-third the total required capacity than a 32-inch pump. In this problem, therefore, three 30-inch pumps will probably give the best operating efficiency.

Assuming that the end of the suction pipe is enlarged to a diameter of 60 inches so that the entrance velocity will be 2.5 feet per second, that the discharge end is enlarged to about 42 inches in diameter so that the discharge velocity will be about 5 feet per second, that one elbow is used, and that the combined

lengths of the pipes is 150 feet, the loss of head will be about as follows:

	Feet
Head lost at entrance	0.1
Head lost at discharge	0.4
Head lost through elbow	1.0
Head lost through pipe friction	1.2
Total	2.7

Calling the sum of these losses 3 feet, and adding this amount to the maximum static head of 20 feet, gives a maximum dynamic head of 23 feet. For the average static head of 10 feet, the dynamic head will be 13 feet.

The water horsepower required to discharge 151 cubic feet per second against a dynamic head of 23 feet is:

$$\text{Water horsepower} = \frac{23 \times 151 \times 62.5}{550} = 394.5$$

Since three pumps are to be used, each pump must develop approximately 132 horsepower. The brake horsepower which must be delivered to the pump shaft will be greater than this, depending upon the efficiency of the pump at maximum head. Since the average dynamic head is 13 feet, the pump will be designed to give its maximum efficiency at about this head. At the 23-foot head, the pumps will not operate at maximum efficiency. Just what this efficiency will be will depend on the design of the pump, and it can be obtained from the data furnished by the manufacturer. It should be not less than 65 per cent, however, and probably the manufacturer would guarantee an efficiency of 70 per cent. The impeller of the pump can be so designed as to give high efficiency both at low heads and low speeds and at high heads and high speeds. Assuming that the pump will operate at 70 per cent efficiency at the maximum head, the brake horsepower required at the shaft of each pump would be 132 divided by 0.70, or 188.

Although each engine must develop 188 brake horsepower when the head is a maximum, the maximum head is so seldom pumped against that it is not necessary to use engines of 188 brake horsepower at full load. Instead, smaller engines can be used and worked at an overload on those occasions when the head is a maximum. A good engine should be able to carry an overload of 25 per cent without an increase of speed and with

good steam economy. Hence, in this problem, engines with a normal rating of 150 brake horsepower will be used.

As illustrated in Fig. 96, the speed of the pump must be increased as the head increases in order to operate at high efficiency. The prime mover, therefore, should be such that its speed can be changed to correspond to the head pumped against and to the load thrown on the engine. In this respect, the steam engine and the oil engine are superior to the electric motor, since their speeds are easily regulated. These engines can,

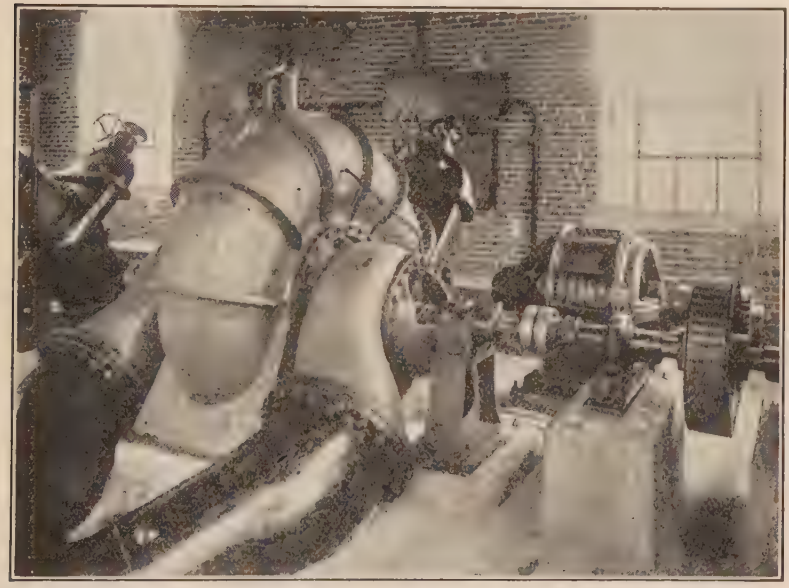


FIG. 102.—30-in. double-suction centrifugal pump of the Marion County Drainage District, Missouri.

therefore, be directly connected to the pump shaft, and thus avoid the losses incident to the belt-driven motor. Where the variation in head is such that the pump can be operated with satisfactory efficiency using only two speeds, the electric motor with chain drive is equally applicable, provided transmission lines are near at hand. A study of the characteristic curves of the pump will determine the range of engine speeds required. The speed will depend upon the design of the impeller.

The kind of prime mover most applicable in this problem will depend upon the funds available for construction of plant, the importance attached to low operating costs, the availability and

price of fuel or electric power, and the operating conditions peculiar to the locality.

If steam is used, high-speed automatic engines directly connected to the pumps should be used. This will require three boilers, which should be of the return-tubular type and with rated capacity of 125 horsepower each.

If internal combustion engines are used, they should be of the horizontal explosion engine, two-stroke cycle type. Each engine unit should consist of two 75-horsepower cylinders.

FIG. 103.—Pumping plant of the Fairfax Drainage District, Kansas City, Kansas.

If electricity is used as a source of power, three 3-phase, induction motors of the wound-rotor type of 150 horsepower should be chosen, with two fixed sprockets on the motor shaft of different size, so that the speed of the pump can be changed when pumping against high heads. Figure 102 shows the silent chain drive on one of the 30-inch double-suction centrifugal pumps of the Marion County Drainage District above Hannibal, Missouri. The two pumps of this district are driven by two 100-horsepower, three-phase, twenty-five-cycle, 440-volt motors running at 480 revolutions per minute. The pumps operate at two speeds, depending on the head. Speed is varied by changing the size

of the chain pinion on the motor shaft. For static heads up to 10 feet, the smaller sprocket is used, delivering from about 25,000 to 15,000 gallons per minute. When the static head is from 10 to 14 feet, which rarely occurs, the larger sprocket for higher speed is used, which maintains an equal range of capacity. The efficiency varies from about 65 to 80 per cent for this variation in head.

Figure 103 shows the pumping machinery of the Fairfax Drainage District, Kansas City, Kansas, installed in 1924. There are five 42-inch double-suction centrifugal pumps, each designed to discharge 50,000 gallons per minute against a total head of 14 feet, when running at 300 revolutions per minute at an efficiency of 80 per cent; or to discharge 39,000 gallons per minute against a total head of 19.2 feet when operating at a speed of 247 revolutions per minute at an efficiency of 82 per cent.

OPERATION OF PUMPING PLANT

Drainage pumping plants differ from other pumping plants in their method of operation. Whereas other plants pump more or less continuously throughout the year, drainage plants are run only a small part of the time and very intermittently. They must be ready at all times for immediate operation, but are actually used only from 10 to 20 per cent of the time. In Southern Louisiana the total time of pumping during the year rarely exceeds 45 days[1] of 24 hours each and often drops as low as 15 days. The total number of days on which pumps are operated average about 70. Along the Illinois River, pumping plants run only from 60 to 90 days of 24 hours, and in some years much less than this.

The method of operating the pumps is important since upon it will depend largely the required capacity of the plant and the satisfactory removal of the excess water. During the fall months, in Illinois, after the crops are harvested, the pumps are not used. During the winter, the pumps should be operated occasionally to remove the water which accumulates during the winter storms. In the late winter and early spring, the pumps should be operated continuously (not necessarily at full capacity) to remove the water from the saturated soil and place the soil in

[1] *Bull.* 71, Office of Experiment Stations, United States Department of Agriculture, p. 66.

condition for early plowing and planting. During the latter part of this period, the pumps will be run more or less intermittently, and it is at this time that the advantage of oil engines or of electric motors is most apparent, since they can be operated at will without loss of time or waste of power. During the growing season, pumping will be required after each storm period, so as to keep the ground water at the desired elevation for the best growth of the crops. It is at this time that the full capacity of the plant may be needed, since the water must be removed before the crops are damaged. During the summer, evaporation and transpiration losses are large, and no pumping is done except after exceptional storms.

It is more economical of power to pump continuously for several days with a small pump operating against a fairly constant head than it is to use pumps so large that the water stored in the suction bay and ditches is so lowered after a few hours of pumping that the pumps must be stopped until the water accumulates again. Hence, the advantage of several small pumping units over one large unit.

COST OF PUMPING PLANTS

The first cost of a pumping plant is made up of the following items—foundation, pumps and accessories, engines or motors, boilers for steam engines, and building for housing the machinery. The cost of equipment is more or less fixed, depending upon the size and power of the units, but the cost of foundations for different plants will vary with the nature of the foundation soil and with the permanency of the structure. If properly designed and constructed, the foundation should outlast several installations of pumps and engines. In many cases the cost of the foundation is a large part of the entire cost of the plant. Also, there is considerable variation in the cost of building. In warm climates, a steel frame building covered with galvanized iron is used. This is a common type of construction used in Louisiana. In Illinois and Iowa, most of the buildings are constructed of brick with concrete foundation walls, supported on piles.

The size, first cost, and year constructed of 16 plants in Louisiana, one in Texas, and one in South Carolina are given in Table XLIV. Plants 1, 2, and 3 are medium-grade steam plants using slide-valve, non-condensing engines; plants 4 and 5 have

Corliss non-condensing engines; plant 6 has a cross-compound Corliss engine; plant 7 has a compound condensing poppet-valve engine using superheated steam; plants 8 to 12, inclusive, have internal combustion engines with a four-stroke cycle and use kerosene or distillate; plants 13 and 14 use explosion oil engines having a four-stroke cycle; plants 15 to 19, inclusive, use

TABLE XLIV.[1]—COST OF PUMPING PLANTS

Name of plant	Acres	Cost	Cost per acre	Year constructed
Phillips Land Company	2,500	$15,000	$ 6.00	1911
Subdistrict No. 1, Lafourche Drainage District No. 6	1,880	10,000	5.32	1912
Subdistrict No. 3, Lafourche Drainage District No. 12	2,250	13,500	6.00	1910
Jefferson Drainage District No. 3	5,000	28,000	5.60	1912
Subdistrict No. 1, Gueydan Drainage District	7,500	40,000	5.32	1912
Subdistrict No. 2, Avoca Drainage District	4,350	34,000	7.81	1911
Subdistrict No. 3, Avoca Drainage District	11,250	73,000	6.48	1913
Subdistrict No. 4, Jefferson Drainage District No. 4	1,800	18,000	10.00	1915
Fayport Subdistrict No. 1, Lafourche Drainage District No. 9	2,000	14,000	7.00	
Dalcour Drainage District	650	10,000	15.40[2]	1913
Subdistrict No. 1, Lafourche Drainage District No. 12	835	10,500	12.57	1915
Subdistrict No. 2, Lafourche Drainage District No. 12	940	12,500	13.30	1915
Subdistrict No. 4, Lafourche Drainage District No. 12	4,240	31,500	7.42	1913
Little Woods Tract	6,943	37,500	5.39	1913
Port Arthur	5,720	54,290	9.50[3]	1918
Ferre Relift	.....	20,000	.....	1918
Richard Relift	.....	15,000	.....	1918
Hine's Relift	.....	8,000	.....	1918

[1] Reproduced from GREGORY, W. B., "Tests of Drainage Pumping Plants in the Southern States," *Bull.* 1067, p. 48, Bureau of Public Roads, United States Department of Agriculture.

[2] Acreage will be considerably increased later.

[3] Plant costs more than it would for an agricultural proposition, as two units must lift water 11 feet if occasion demands it.

TABLE XLV.[1]—COSTS OF OPERATING DRAINAGE PUMPING PLANTS OF VARIOUS TYPES IN LOUISIANA, SOUTH CAROLINA, AND TEXAS

Interest and Depreciation Not Considered

No.	Name of plant	State	Area drained, acres	Plant capacity				Area from which plant will remove 1.25 inches per day, acres[2]	Assumed cost of oil per gallon, cents	Yearly cost of operation					Yearly cost per acre[3]	
				Basis 10 second-feet nozzle velocity		From test at 5-foot lift				Fuel to lift 29 acre-inches 5 feet	Fuel for starting, lubricating oil, and supplies	Total fuel and supplies	Labor	Total yearly cost of operation	Total fuel and supplies	Total fuel, supplies, and labor
				Second-feet	Acre-inches per day	Second-feet	Acre-inches per day									
1	Phillips Land Company	Louisiana	2,500	102.1	0.97	108.0	1.03	2,060	3.5	$1,288	$321	$1,609	$1,320	$2,929	$0.78	$1.42
2	Subdistrict No. 1, Lafourche Drainage District No. 6	Louisiana	1,880	62.8	0.80	52.0	0.66	990	3.5	617	155	772	1,320	2,092	0.78	2.11
3	Subdistrict No. 3, Lafourche Drainage District No. 12	Louisiana	2,250	98.2	1.04	78.0	0.83	1,480	3.5	924	231	1,155	1,320	2,475	0.78	1.67
4	Jefferson Drainage District No. 3	Louisiana	5,000	251.4	1.20	238.0	1.13	4,530	3.5	1,775	445	2,220	1,740	3,960	0.49	0.87
5	Subdistrict No. 1, Gueydan Drainage District	Louisiana	7,500	318.0	1.01	365.0	1.16	6,950	3.5	2,725	680	3,405	1,740	5,145	0.49	0.74
6	Subdistrict No. 2, Avoca Drainage District	Louisiana	4,350	125.7	0.69	127.7	0.70	2,420	3.5	774	194	968	2,220	3,188	0.40	1.32
7	Subdistrict No. 3, Avoca Drainage District	Louisiana	11,250	282.7	0.60	256.0	0.54	4,870	3.5	1,052	236	1,315	2,220	3,535	0.27	0.73
8	Subdistrict No. 4, Jefferson Drainage District No. 4	Louisiana	1,800	98.2	1.30	91.0	1.20	1,730	9.0	540	135	675	1,500	2,175	0.39	1.26
9	Subdistrict No. 1, Lafourche Drainage District No. 9	Louisiana	1,760	98.2	1.33	39.2[5]	0.52	747	9.0	233	58	291	1,500	1,791	0.39	2.40

10	Dalcour Drainage District	Louisiana	650[4]													
11	Subdistrict No. 1, Lafourche Drainage District No. 20.....	Louisiana	2,460	141.4	1.36	192.0	1.86	3,650	9.0	1,140	286	1,426	1,500	2,926	0.39	0.80
12	Combahee Drainage District...	South Carolina	1,950	80.5	0.98	79.4	0.97	1,512	9.0	472	118	590	1,500	2,090	0.39	1.38
13	Subdistrict No. 1, Lafourche Drainage District No. 12.....	Louisiana	835	62.8	1.79	59.0	1.68	1,123	6.5	252	63	315	1,500	1,815	0.28	1.61
14	Subdistrict No. 2, Lafourche Drainage District.....	Louisiana	940	62.8	1.59	42.0[5]	1.06	800	6.5	179	45	224	1,500	1,724	0.28	2.16
	Louisiana Irrigation & Mill Company.[6]															
15	Jamison Relift[6].....	Louisiana	.....	141.4	....	71.4	....	1,350	6.5	302	76	378	1,500	1,878	0.28	1.39
16	Ferre Relift[6].....	Louisiana	.....	62.8	....	84.7	....	1,613	6.5	362	90	452	1,500	1,952	0.28	1.21
17	Richard Relift[6].....	Louisiana	.....	62.8	....	120.0	....	2,286	6.5	512	128	640	1,500	2,140	0.28	0.94
18	Hine's Relift[6].....	Louisiana	.....	31.4	....	45.6	....	870	6.5	195	49	244	1,500	1,744	0.28	2.01
19	Port Arthur.....	Texas	2,175	.....	....	281.6	....	5,360	6.5	1,200	300	1,500	1,500	3,000	0.28	0.56
20	Subdistrict No. 4, Lafourche Drainage District No. 12.....	Louisiana	4,240	251.4	1.41	243.0	1.37	4,630	3.5	334	83	417	1,500	1,917	0.09	0.41
21	Station No. 1, New Orleans....	Louisiana														
22	Little Woods Tract.....	Louisiana	6,943	292.0	1.00	364.0	1.25	6,940								

[1] Reproduced from *Bull.* 1067, pp. 46–47, Bureau of Public Roads, United States Department of Agriculture.
[2] Quantities from tests.
[3] Based on acreage from which plant will remove 1.25 inches in 24 hours.
[4] Plant incomplete.
[5] Weeds in pump; see test.
[6] Low-lift irrigation plants.

TABLE XLVI.—COST OF OPERATION OF PUMPING PLANTS

District	Number of years' operation	Average cost per acre per year in dollars											
		Labor		Fuel		Repairs		Supplies		Superintendence		Total	
		A.[1]	T.[2]	A.[1]	T.[2]	A.[1]	T.[2]	A.[1]	T.[2]	A.[1]	T.[2]	A.[1]	T.[2]
I. Steam-driven Plants													
Nutwood	5(1914)	0.115	0.082	0.158	0.113	0.015	0.011	0.038	0.027	0.079	0.057	0.405	0.290
Eldred	1(1914)	0.129	0.126	0.153	0.150	0.019	0.017	0.028	0.027	.....	.....	0.329	0.320
Hillview	5(1913)	0.190	0.126	0.278	0.185	0.047	0.031	0.032	0.021	0.114	0.076	0.661	0.439
Big Swan	7(1913)	0.123	0.098	0.250	0.200	0.032	0.026	0.031	0.025	0.071	0.056	0.507	0.405
Meredosia	9(1914)	0.100	0.069	0.314	0.238	0.023	0.017	0.020	0.015	0.073	0.055	0.530	0.394
Crane Creek	4(1914)	0.173	0.154	0.281	0.250	0.014	0.012	0.049	0.043	.....	.....	0.517	0.459
Coal Creek	5(1913)	0.331	0.300	0.598	0.540	0.049	0.044	0.041	0.037	0.012[3]	0.010[3]	1.031	0.931
Lacey	4(1905)	0.072	0.072	0.247	0.247	0.030	0.030	0.017	0.017	.....	.....	0.366	0.366
Lacey	2(1910)	0.151	0.151	0.416	0.416	0.017	0.017	0.027	0.027	.....	.....	0.611	0.611
Spring Lake	5(1914)	0.181	0.103	0.306	0.175	0.024	0.014	0.081	0.046	0.138	0.079	0.730	0.417
Louisa-Des Moines Iowa	2(1913)	0.164	0.133	0.204	0.165	0.041	0.033	0.027	0.022	0.048	0.039	0.484	0.392
Des Moines County No. 1	2(1913)	0.140	0.053	0.162	0.062	0.034	0.013	0.033	0.012	0.036	0.014	0.405	0.154
Average	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	0.432
II. Electrically Driven Plants													
Big Swan	1(1914)	0.116	0.098	0.592	0.473	0.000	0.000	0.000	0.000	0.071	0.056	0.779	0.627
Lacey	2(1914)	.....	.....	0.740[4]	0.740[4]	.....	.....	.....	.....	.....	.....	0.740	0.740
Pekin-LaMarsh	3(1914)	0.132	0.107	1.160[4]	0.940[4]	0.021	0.018	0.002	0.002	.....	.....	1.315	1.067
Coal Creek	1(1914)	0.159	0.143	0.860	0.776	0.000	0.000	0.010	0.010	0.020	0.019	1.049	0.947
East Peoria	1(1914)	0.504	0.244	1.200	0.582	0.001	0.001	0.023	0.011	0.372	0.180	2.100	1.018
Average	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	0.880

[1] Based on assessable area.
[2] Based on total drainage area.
[3] Partial.
[4] Cost of power calculated on basis of schedule in effect at the Coal Creek Plant.

TABLE XLVII.—TOTAL COSTS OF PUMPING PLANTS

District	Horsepower of engines	First cost	Annual fixed charges	Annual operating expenses	Assessed area	Watershed area	Charges per year per acre on				Total charge per acre per year on	
							Assessed area		Watershed area			
							Fixed	Operating	Fixed	Operating	Assessed area	Watershed area
I. Steam-driven Plants												
Nutwood	500	$ 36,480	$ 4,750	$ 4,338	10,700	15,000	$0.444	$0.405	$0.317	$0.290	$0.849	$0.607
Eldred	300	28,000	3,640	3,132	9,305	9,500	0.391	0.337	0.383	0.330	0.728	0.713
Hillview	300	25,000	3,250	8,140	12,300	18,500	0.264	0.662	0.176	0.440	0.926	0.616
Big Swan	300	25,000	3,250	6,084	12,000	15,000	0.271	0.507	0.217	0.406	0.778	0.623
Meredosia	200	18,000	2,470	1,995	3,760	5,000	0.657	0.531	0.494	0.399	1.188	0.893
Crane Creek	250	16,000	2,080	2,759	5,340	6,000	0.390	0.516	0.347	0.460	0.906	0.807
Coal Creek	200	16,000	2,080	6,912	6,700	7,426	0.310	1.032	0.280	0.930	1.342	1.210
Lacey	220	15,000	1,950	1,898	5,200	5,200	0.375	0.365	0.375	0.365	0.740	0.740
Lacey	220	15,000	1,950	1,950	3,200	3,200	0.610	0.610	0.610	0.610	1.220	1.220
Louisa-Des Moines	500	50,000	6,500	6,280	13,000	16,000	0.500	0.483	0.406	0.393	0.983	0.799
Spring Lake	600	60,000	7,800	8,600	12,850	22,500	0.607	0.670	0.347	0.382	1.277	0.729
Des Moines County No. 1	1,230	144,000	18,700	11,570	28,600	75,000	0.654	0.405	0.249	0.155	1.059	0.404
Average	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	0.780
II. Electrically Driven Plants												
Big Swan	400[1]	20,000	2,600	10,790	12,000	15,000	0.217	0.900	0.173	0.720	1.117	0.893
Lacey	185	10,000	1,300	3,200	3,200	3,200	0.406	1.000	0.406	1.000	1.406	1.406
Pekin-LaMarsh	135	7,800	1,000	3,520	2,600	3,200	0.385	1.354	0.313	1.100	1.739	1.413
Coal Creek	300	20,000	2,600	7,027	6,700	7,426	0.388	1.049	0.350	0.946	1.437	1.296
East Peoria	125	10,500	1,360	1,570	750	1,550	1.810	2.093	0.877	1.013	3.903	1.890
Average	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	1.380

[1] Plant has one 100-horsepower motor and 300 horsepower steam engine. First cost estimated as if four 100-horsepower motors were installed.

explosion oil engines having a two-stroke cycle; plant 20 has a two-stroke Diesel engine; and plants 21 and 22 (Table XLV) use electric motors. Most of these plants have a pumping capacity of from 1 to 1¼ inches in 24 hours. In using these cost data, the year of construction must be noted and allowance made for the present higher prices of materials and machinery.

TABLE XLVIII.—AMOUNTS PUMPED AND UNIT COSTS OF PUMPING

District	Year	Amount pumped, acre-feet	Average static head, feet	Cost of lifting 1 acre-foot 1 foot in dollars
Louisa-Des Moines.....	1910	15,900	4.20	0.086
Louisa-Des Moines.....	1911	15,000	2.82	0.141
Louisa Des Moines.....	1912	20,132	5.20	0.064
Louisa-Des Moines.....	1913	16,002	5.74	0.063
Coal Creek.....	1914	6,800	9.40	0.110
East Peoria.....	1914	901	7.02	0.158

Table XLV gives additional data for these plants, including the annual cost of operation. These operating costs, however, do not include depreciation of plant, an item which should be considered in the annual cost. The curves in Fig. 104 represent the average total annual cost, including depreciation, of the several plants, grouped as to type of motive power.

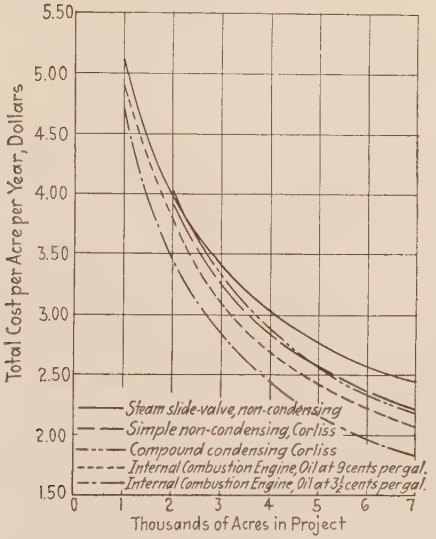
Table XLVI[1] gives the cost of operation of 14 pumping plants on the Illinois River and two plants on the Mississippi River in Iowa. It is to be noted that the costs given are for operation only and do not include interest, taxes and insurance, nor depreciation.

Table XLVII[1] gives for the same districts the horsepower of engines, first cost of plant, and total annual costs, including fixed charges. The fixed charges consist of interest on investment at 6 per cent, depreciation of plant at 6 per cent, and taxes and insurance at 1 per cent.

The normal annual amount of water pumped in the districts is about 16 inches. The actual amounts pumped in the three districts for which the most accurate pumping records are available, the average static head, and the cost of lifting 1 acre-foot 1 foot are shown in Table XLVIII.

[1] Bulletin No. 71, Office of Experiment Stations, U. S. Dept. of Agriculture.

It will be observed that the first costs of the plants given in Table XLVII vary from $70 to $100 an engine horsepower. These are pre-war prices; at the present time (1924) the cost of a well-constructed pumping plant will be from $175 to $200 an



(Reproduced from "*Bulletin* No. 1067," p. 53. U. S. Dept. of Agriculture.)

FIG. 104.—Total cost per acre per year, of drainage pumping plants in Louisiana, including fuel, supplies, labor, and fixed charges.

engine horsepower. This approximate value may be used in making a preliminary estimate; but for the final estimate, all the items of the design should be considered separately and their costs determined by quotation on materials and equipment and by estimation of the labor costs, based on current prices.

CHAPTER XI

FLOOD PROTECTION BY CHANNEL IMPROVEMENT

The purpose of channel improvement is to increase the discharge or carrying capacity of a stream so as to enable the water to flow off faster and thus decrease the height and duration of floods and reduce the frequency of flood damage.

There are two ways in which the discharge of a stream can be increased: (1) by increasing the cross-section or size of the channel; and (2) by increasing the velocity or rate of flow of the water.

Increasing the Channel Cross-section.—It is not feasible usually to enlarge the channel of a stream so as to retain all the flood waters within its banks. If the stream is narrow and shallow, its watershed small, the depth of flooding small, and the period of overflow short, it may be practicable to deepen and widen the channel to a limited extent and to use the excavated material to raise the height of the banks. But for streams draining several thousand square miles, it is economically impossible to solve the flood problem in this way, since the volume of flood water is so large that it would require a channel from 500 to 1,000 feet in width to pass the flood waters within its banks. Obviously, the cost of such enlargement would be many times the value of the protection received; and further, the desired protection can be secured in an easier and cheaper way—the construction of levees. The profitable limit of channel enlargement—at least for any considerable length of channel—is generally conceded to be from 100 to 125 feet.

Increase in Velocity.—There are three factors which affect the velocity of flowing water. These are: (1) the slope of the water surface; (2) the hydraulic radius; and (3) the roughness of the channel. These factors will be discussed in reverse order.

Roughness of Channel.—The roughness factor includes all obstacles which retard the flow. In large streams the percentage of the channel cross-section which is obstructed by trees and weeds on, or near, the banks is small, and the reduction of the

mean velocity and cross-section is negligible; but for small streams, the reverse is true. Further, the channels of large streams are not so likely to become obstructed by the lodgement of floating trees and brush, whereas in the case of small streams this is almost certain to occur. Any obstacle of this nature backs up the water and reduces the slope and the velocity. In general it may be stated that little is gained by clearing the banks of large streams and removing the obstruction which may have formed in the channel; but in the case of small streams, it frequently happens that the efficiency of the channel can be increased from one-third to one-half by clearing the banks and removing fallen trees, snags, and sandbars from the channel. In fact, no greater return can be obtained for the amount expended by any other form of improvement.

The roughness factor also includes the checking of the flow of the water at sharp bends in the channel, and, where such bends can be eliminated by short cut-offs, it is usually profitable to construct them. The construction of cut-offs, however, introduces other elements which will be considered later.

Hydraulic Radius.—While this is an important factor when the velocities at different stages of a stream are considered, yet during flood periods when the entire channel is full of water its effect is not so marked. The velocity increases as the hydraulic radius increases, as shown in Fig. 60. The hydraulic radius of a channel at bankful stage can be increased by either deepening or widening the channel; but for the same enlargement of channel, deepening is more effective than widening.

For example, suppose the bottom width of a channel is 50 feet, the slope of the banks 1 on 1, and the depth of flow 8 feet. If the bottom of the channel is lowered 4 feet by dredging, the slope remaining the same, the bottom width would be 42 feet and the depth of flow 12 feet, and the channel cross-section would be enlarged 184 square feet. This deepening of the channel would increase the hydraulic radius from 6.39 to 8.53 feet and cause the water to flow about 22 per cent faster, depending upon the roughness factor.

If the original 50-foot channel, instead of being deepened, were widened to 73 feet, the enlargement of the channel would again be 184 square feet, but the hydraulic radius would be increased thereby to only 6.78 feet, and the velocity would be increased only about 4 per cent as compared to 22 per cent in the

former case. Hence, it is seen that increasing the depth has a much greater effect upon the velocity than increasing the width.

Since the outlet of a stream cannot be lowered in most instances, the deepening of the channel upstream would decrease the slope of the stream, and the loss of velocity due to the flatter slope would usually be greater than the increase of velocity due to the greater hydraulic radius. If, however, the stream can be shortened by the construction of cut-offs, the channel can be deepened and the slope increased at the same time; hence, where channel improvement includes enough cut-offs to make possible an increase of depth in the used portions of the old channel, the hydraulic radius is an important factor.

The hydraulic radius can also be increased by raising the banks by levees instead of lowering the bed by dredging. Since, however, the cheapest method of constructing levees on the banks is by dredging the material from the bed, these two methods of increasing the hydraulic radius are generally considered as one method.

The enlargement of a channel by widening is beneficial in increasing the cross-section and thereby the discharge, but the increase in the hydraulic radius is negligible.

Slope of Stream.—The velocity of flowing water varies as the square root of the slope. For example, to double the velocity it is necessary to increase the slope fourfold.

Since the total fall between any two points on a stream is constant, the only way in which the slope, or fall per unit length, can be increased is by shortening the channel of the stream. As most of the streams subject to overflow wind back and forth across their flood plains, they are readily susceptible to shortening. The shortening is accomplished by the construction of cut-offs across the narrow necks of land made by sharp loops in the channel. The lengths of these cut-offs vary from about one-half to possibly one-tenth of the length of the old channel. The purpose of cut-offs is threefold: (1) to increase the velocity by increasing the slope; (2) to shorten the distance which the water has to travel; and (3) to decrease the length of levees necessary to confine the flood waters—in case levees are to be used as a part of the flood protection works.

When considered in stretches of 50 to 100 miles or more, the crooked channels of many streams are about twice the length of the valleys in which they flow, due to the meandering of the streams

across their flood plains. Assuming that the length of channel could be cut in half by the construction of cut-offs, the increase in velocity due to the increased slope would be about 40 per cent; and this increase together with the shorter path provided for the water would enable water from the upper end of the watershed to reach the lower end in a much shorter time. Whether or not the benefits to be derived from the maximum straightening justify the expenditure incident thereto and, if not, to what extent cut-offs are justifiable, depends on the size of the stream, the size and arrangement of the tributary streams, and the width of the flood plain to be protected from overflow. Varying conditions give rise to three general cases, as follows:

Case 1. *Narrow Flood Plains.*—Where the flood plain between the stream and the high ground is less than about ½ mile, it is generally considered that the cost of levees far enough from the stream to provide a floodway for the flood waters is greater than the benefits to the land protected. In this case, the flood protection work consists entirely of channel improvement, and it is here that the largest amount of shortening is justifiable. There may be, however, a few pockets of land here and there in the valley of sufficient width to warrant additional protection by levees.

Even in a crooked channel there are some fairly straight stretches, and these portions should be utilized as far as possible in the new plan. Sometimes, the straight stretches of old channel require very little dredging to make them conform to the desired cross-section and slope; on the other hand, the cost of dredging in the old channel may be as much, or even more than, the cost of a new channel, especially so if the old channel is full of snags and buried logs and the banks are covered with large, willow trees.

Where the old and new channels cross each other, the difficulties of construction are increased, since in leaving the old channel a dam must be constructed below the point to pond the water and enable the dredge boat to approach properly the bank to be cut; and many contractors prefer to dig an entirely new channel where such crossings are frequent.

Existing highway and railroad bridges are generally controlling points, since the cost of new bridges add considerably to the cost of the project, and it is seldom that such additional cost is warranted.

Another important factor is the human element. Under the laws of most states, channel improvement is undertaken by the landowners affected, and the cost is paid by them. This means that at least a majority—and usually a large majority—of the landowners must not only be in favor of flood protection, but must also approve the plans prepared by the engineer. The landowner's chief concern is the effect which the improvement will have upon his land; and if the proposed route cuts through his best land, dividing it into tracts which are more difficult of access and more difficult to cultivate, he will generally oppose the project. Because of the objection of some large and influential landowner, the best route from an engineering viewpoint may have to be abandoned.

As an example of streams which fall under this classification, a portion of the Sangamon River in Illinois which is now being improved (1924) is shown in Fig. 105. The channel improvement starts at the highway bridge south of Niantic and extends west to a point about 1 mile west of the highway bridge north of Roby. The distance by the old channel is about 21.8 miles, and the length of the straightened channel is 15.8 miles. The new channel has a bottom width of 40 feet for the upper 7 miles and a width of 50 feet for the lower end. The slope throughout the improvement is 0.033 per cent. The material excavated from the channel is used to raise the height of the banks. The waste banks are about 8 feet high and are placed back from the edge of the ditch a distance of from 12 to 16 feet.

Without doubt this channel will erode during each flood period and in time will become much wider. Because of erosion, the waste banks should be placed as far from the channel as the limitations of the dredging equipment will permit.

Case 2. *Wide Flood Plains.*—In the case of large streams which naturally have wide flood plains, flood protection is never attempted by stream straightening alone, because the volume of flood water is so great that the increased discharge caused by the cut-offs would be negligible as far as lowering flood heights is concerned. The universally accepted method of flood protection for large streams is the construction of levees far enough from the river banks to provide an adequate channel for the flood waters.

Not only are cut-offs ineffective in reducing flood heights in a large stream—unless the entire flood flow is confined between

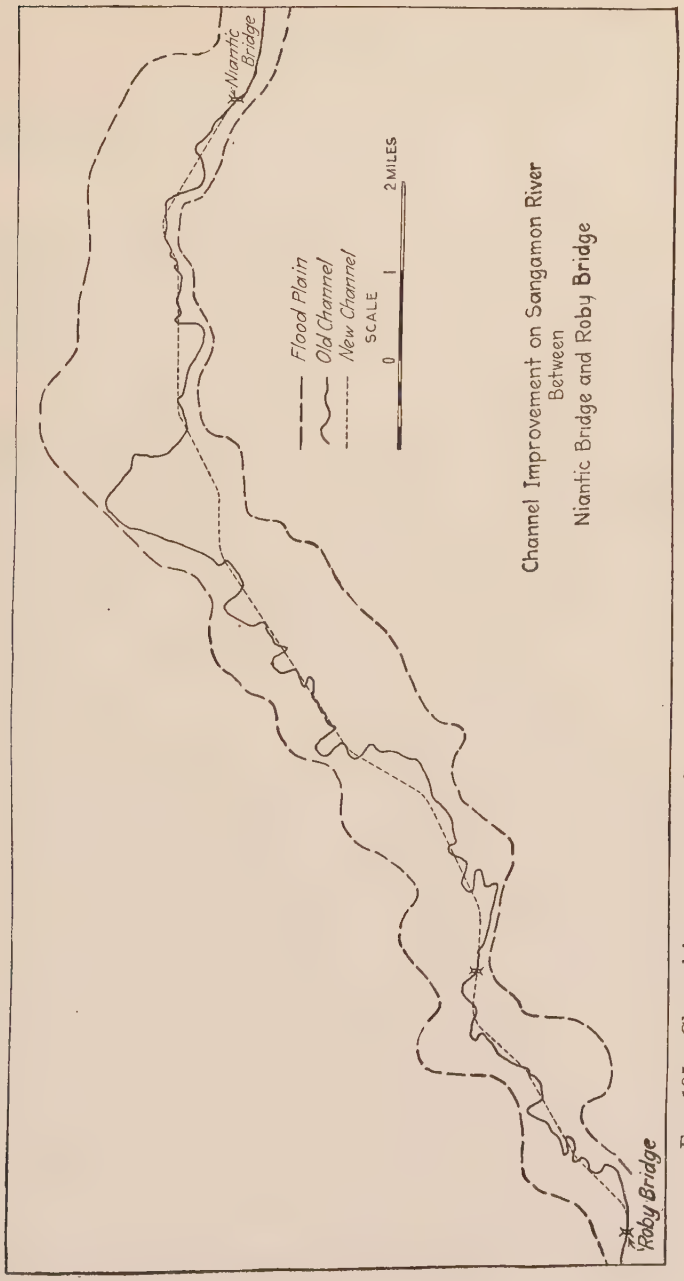


FIG. 105.—Channel improvement on Sangamon River between Niantic bridge and Roby bridge, Illinois.

levees—but also they are considered a decided menace on alluvial streams, because of the disastrous effect upon the regimen of the stream. The water flowing from the lower end of a cut-off will not have the same direction as it did before the cut-off was made, and as a result the water will attack the banks at a different place and with much greater force.

The effect of cut-offs on the Mississippi River has been a disturbance of the regimen of the stream for more than 100 miles both above and below the cut-offs, and the washing away of levees and farm lands below. The policy of the Mississippi River Commission has been to prevent natural cut-offs by the protection of the banks at critical points. Other effects of cut-offs will be discussed later.

Case 3. *Flood Plains of Medium Width.*—Under this classification fall those streams which have a width of about 200 feet at bankful stage and a flood plain of from 1 to 4 miles in width. Since the flood flow of a river is indicated by the flood plain which it has made for itself, it is evident that for streams of this class complete flood protection cannot be secured by channel improvement. In this case, as in the former one, levees or artificial banks must be constructed to protect the land from overflow. The flood plains of these streams are wide enough so that, even after a portion has been taken for a floodway, there will remain sufficient land between the levees and the bluffs to justify the cost of levee construction. A certain amount of channel improvement, nevertheless, is generally advisable for these streams for several reasons: (1) the construction of cut-offs at the most pronounced bends rectifies the channel and makes straighter and shorter levees possible; (2) the increased capacity of the channel may eliminate minor floods and enable the floodway to be used to some extent; (3) by channel rectification in places, the amount of land which can be protected by a given length of levee may be increased; and (4) the straighter the levees, the less likely are they to be undermined or washed away by the current.

The Kaskaskia River in southern Illinois may be cited as a typical example of this class of stream. Figure 106 shows a portion of this river with a recommended plan of flood protection. The stream has a width of from 150 to 300 feet at bankful stage and a depth of from 15 to 22 feet. The slope of the bed is about 1 foot per mile through this stretch. As shown in Fig. 106, the bottom lands are protected from overflow by levees

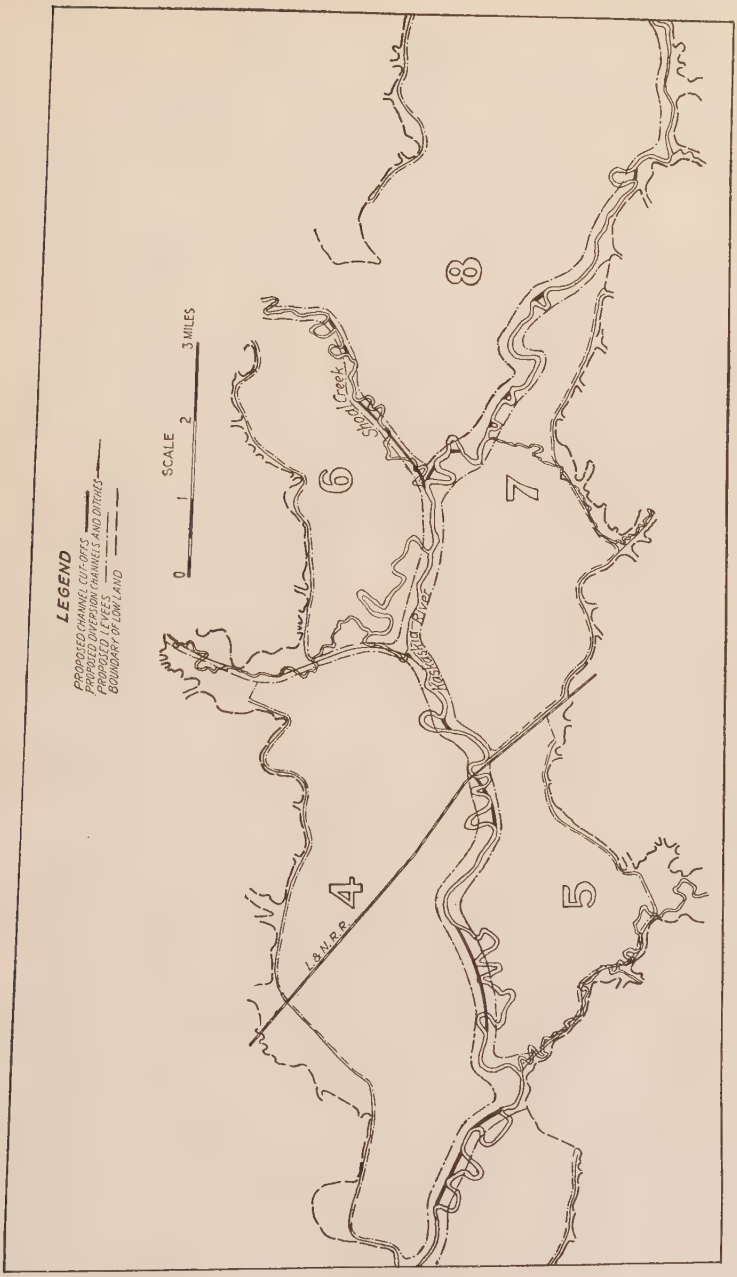


FIG. 106.—Proposed method of reclamation of overflow lands along Kaskaskia River.

which form a floodway from 1,200 to 1,400 feet in width. Several large creeks enter the river in this portion, and, since these must also be leveed back to the bluffs, the land is naturally divided into tracts which can be reclaimed independently as units. As may be seen on the map, the channels of the creeks from the bluffs to the river are extremely crooked, and in most cases entirely new channels are provided. A large amount of hill water enters the valley from the bluffs, and diversion ditches with levees on the river side are included in the plan of reclamation.

Cut-offs are recommended only where a considerable rectification of the channel and floodway is possible at small expense. In every case, the saving in the cost of levees is much more than the cost of the cut-offs.

Effect of a Single Cut-off.—In Fig. 107 is shown a portion of a stream with a sharp bend, *BCD*. Now suppose that by the construction of the cut-off, *BD*, shown by dotted lines, the channel distance between *B* and *D* is reduced to one-third its former length, and that the velocity of the water through the new channel at bankful stage is doubled, due to the greater slope and the smaller roughness factor. What effect will this cut-off have upon the flow?

Steady Flow.—If it is assumed that steady flow exists in the channel, that is, that the inflow at point *B* is constant, and that the stream is within its banks, the discharge through the cut-off will be the same as through the old channel, since the amount of inflow is the same; but because the water flows much faster through the cut-off channel than through the old channel above and below the cut-off section, the discharge will be carried at a much lower stage. The increased velocity will be attained gradually. The velocity will begin to increase at some point *A* above the head of the cut-off and will reach its maximum at some point *F* in the cut-off section. The section *AF* in which the velocity changes from the old to the new value is called the "drop-off" section, and its length will depend upon all the hydraulic elements of the old and new channels. The change in stage will take place in the drop-off section and the surface of the water will form a curve, concave downward, called the "drop-off curve."

Likewise, at the lower end of the cut-off, there will be a reverse change in both velocity and stage. The curve here is concave

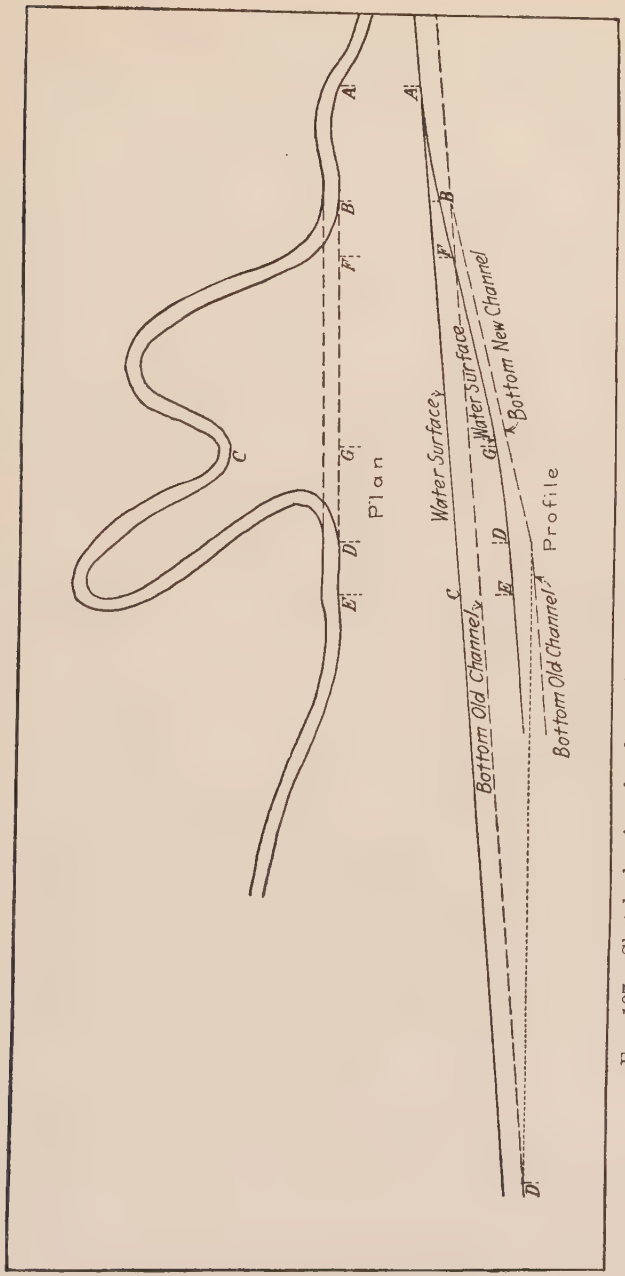


FIG. 107.—Sketch showing the change in profile of channel caused by a single cut-off.

upward and is called the "backwater curve." The velocity will begin to decrease and the stage to increase at some point G in the cut-off section. When the point D is reached, the velocity will still be greater than that above point A and, therefore, the stage at D will be lower than that at A. At some point E—the end of the backwater curve—the velocity and stage will be the same as at A. Thus it is seen that the effect of the cut-off is beneficial so far as stage of stream is concerned for some distance both above and below as well as through the cut-off section. The change in the water surface is shown in the profile in Fig. 107.

Since cut-offs are usually short, the points F and G will usually overlap and the water surface throughout the cut-off section will be curved; and because of this overlapping the water never reaches the maximum velocity corresponding to the slope of the bed and to the hydraulic radius.

Under the assumption of steady flow, the effect of a cut-off is not to increase the discharge and stage of the water below the cut-off, but is simply to lower the stage of the water through the cut-off and for a greater or less distance above and below.

Unsteady Flow.—Steady flow seldom exists in a stream, and during a storm period the inflow is constantly increasing. As the inflow increases, the water will rise until presently the stream at point A in Fig. 107 is flowing bankful. At this stage of the storm period, the stream above point A will be out of its banks; but between points A and E, the water will be carried at less than bankful stage, and through the greater portion of the cut-off section at much less than bankful stage. Below point E the water will be somewhat below the top of the banks, under the assumption that all the inflow comes from above point A. If the inflow at A should remain constant at this stage, in a short time the channel below E will flow full. If the flood crest is reached at this time, the lands below point E will not be flooded as a result of the cut-off.

As the flood wave advances downstream and the inflow continues to increase, presently the cut-off channel will be flowing full. At this stage, the land above point F and that below point G will be flooded; but between F and G there will be no flooding. Hence, up to the time that the capacity of the cut-off channel is exceeded, the bottom land along the cut-off—except for a portion at its lower end—will be protected from the overflow to which it

would have been subject under the old channel conditions. But the flooding below point *E* will be greater than before the cut-off was made, because the water which was formerly stored in the flood plain along *BCD* is now discharged by the cut-off channel, thus increasing the stage of the water below *E*. Also, the time required for the flood crest to reach point *E* is shortened, which means that a storm of shorter duration will cause the stream to overflow its banks at *E*.

It must be understood that the points *A*, *F*, *G*, and *E* are not fixed points, and that they change with the stage of the stream. On a rising stream, the slopes above, through, and below the cut-off section become more nearly the same, and both the drop-off and backwater curves are shortened. Also, the distance *AB* will be much greater than *BF*, and the distance *DE* will be less than *DG*. For streams with flat slopes, the effect of lowering the water at the head of the cut-off is felt many miles upstream; on the other hand, the distance *DE* is comparatively short.

As the storm continues and the flood wave continues to advance, flooding will occur in the cut-off section, but the depth of flooding will be less than that above and below the section, except in those cases where the inflow becomes so much greater than the outflow that the difference in the capacity of the old and new channels is negligible.

Whenever the depth of flooding in the cut-off section is less than that above, the depth of flooding below the section will be greater than under the original conditions. Thus, any benefit which accrues to the land along the cut-off section is counteracted by a damage to the lands below.

The above conclusions are based on the assumption that all the inflow comes from above point *A*. If a tributary stream discharges between *A* and *E*, the conclusions will be somewhat different. This phase is considered later.

Effect of a Series of Cut-offs.—If a series of cut-offs is made and the old more or less straight stretches of channel between the cut-offs are *unimproved*, the conditions are very similar to those described above for a single cut-off. Since the old straight stretches are not improved, no more water can flow through them *at bankful stage* than before the cut-offs were constructed—unless the stretches are very short—and the capacity of the stream as a whole is no greater than before. As long as the slope and cross-section of a given stretch of channel are

unchanged, no amount of channel improvement above will increase the discharge through the given stretch. But, since the distance which the water has to travel is greatly reduced by the construction of cut-offs, as well as the storage capacity of the old channel and flood plains, a flood wave will reach the points downstream in a much shorter time than before the cut-offs were made. Also, since the depth of flooding along the cut-off sections is generally less, the depth of flooding along the unimproved stretches will be greater; and, since the depth of overflow below a cut-off is greater than before, the slope of the water surface through the unimproved sections will be greater and hence the discharge in and above the channel and along the flood plain will be greater than before, which will result in a progressively larger discharge at points downstream. Thus it is seen that so far as flood protection is concerned, the elimination of bends in a channel is of small value—except locally—unless the stretches between the cut-offs are improved. For the improvement to be of real value, the slopes of the old channel sections must be increased so that the velocity through all portions of the stream will be approximately the same. This can be done by lowering the grades through the cut-off sections and using the fall saved thereby in increasing the grades in the stretches above. This statement applies to small streams where the cost of dredging the old channel is not prohibitive.

For the purpose of discussion, assume that a portion of a crooked stream 100 miles in length is improved by the construction of a series of cut-offs and the dredging of the old, straight portions of the channel so that the velocity throughout the stretch is approximately the same. Also, that the improved channel is only 50 miles in length; and further, that the velocity is increased 50 per cent by the improvement. What protection will the improvement give the adjoining land against a flood *entering at the upper end?*

Since the velocity of the water in the improved channel is 50 per cent greater than in the old, crooked channel, the former discharge at the bankful stage will now be carried at considerably less than bankful stage, and it will require a more severe storm or one of longer duration to cause the stream to overflow. Hence, former minor floodings of the bottoms would be eliminated. If the stream should overflow its banks, the rate of increase in the depth of overflow would be less, the high-water stage of the flood

would be lower, the crest of the flood would be reached sooner, and the stream would be within its banks again in a shorter time than under the old channel conditions. This will be the condition along the 50-mile stretch, except at its lower end. If the outlet below is not improved, it cannot handle the increased discharge brought to it through the improved channel, and the result will be a ponding of the water at the lower end of the improvement; and at this point, flooding will be more frequent and more serious than before. The bottom land along the major portion of the 50-mile stretch, and for some distance above, will have been benefited at the expense of the lands at the lower end of the stretch and below it. It is evident, therefore, that channel improvement should extend to the mouth of the stream, so that the increased discharge of the channel may have a free outlet; from which it follows that channel straightening can be applied extensively only in the case of small streams.

Concentration of Tributary Flow.—There is another phase of the problem to be considered and that is the effect of tributary flow. Thus far in the discussion all the inflow has been assumed as concentrated at one point—the upper end of the improvement. Actually, there is inflow at numerous points throughout the length of the stream. It is a well-known fact that the closer the mouths of the tributaries of a stream are to each other, the greater and more frequent are the floods to which the main valley is subject for the same rainfall conditions. This is due to the concentration of the tributary flood waters and to the shorter time required for concentration to take place.

In order that this effect may be better understood, let us consider two watersheds of the same size and with the same rainfall and run-off, but of different shapes. The first watershed is fan-shaped with all the tributaries entering the main stream at approximately the same point. The second watershed is long and narrow, the tributaries entering the main stream at widely separated points throughout the valley. If both watersheds are subjected to storms of equal intensity, the first watershed will suffer more from flooding than the second, because in the first watershed the flood crests of the several tributaries will reach the main stream at about the same time, and the result of the concentrated flow will be the flooding of the main valley; whereas, in the second watershed, the flood water from each tributary will reach the main stream and flow off before the water from the

tributary above has time to reach the tributary below; and there will be no concentration of the flood crests of the several tributaries except in floods of long duration. Fan-shaped watersheds or watersheds where the distances between the mouths of the tributaries are short are, therefore, subject to more frequent flooding than watersheds in which the distances between tributaries are greater.

The shortening of a channel by cut-offs decreases the channel distance between tributaries and, so far as flood flows are concerned, has the effect of changing the shape of the watershed.

In the last example of channel improvement, the length of the old channel was reduced one-half and the velocity of the water was increased 50 per cent. The result would be that water from any point in the valley would reach any other point in from one-third to two-thirds of the time it required before the improvement was made, and the concentration of tributary flow might more than offset the advantage gained by the greater velocity and cause a greater flooding of certain portions of the valley than would have occurred under the original conditions of channel. For example, assume that in an unimproved channel it takes 24 hours for the crest of a flood from the upper end of the stream to reach the mouth of an important tributary downstream. Also, assume that it requires 12 hours for the flood crest of the tributary stream to reach the main stream. Then it will require a storm of at least 24 hours to cause the two flood crests to concentrate and cause a maximum stage in the lower stream. Now if by channel straightening the flood crest from the upper end of the main stream reaches the mouth of the tributary in 12 hours, the critical condition of concentration of flows will be caused by a 12-hour rain instead of a 24-hour rain; and since 12-hour storms are more frequent than 24-hour storms, the valley below the junction will be flooded more frequently than before the improvement was made. Hence, the location of the tributary streams and the time required for their flood waves to reach the main valley are important factors to be considered in making channel improvements.

Distribution of Valley Storage.—The flooding of the bottom land along a stream is caused by the fact that the water enters the valley faster than the channel can discharge it. The flood waters from all parts of the watershed are brought to the main

valley by the tributary streams which discharge their waters at different points in the valley. The improvement of the main channel will have no effect on the total inflow, but it will have a decided effect on the distribution of the water which is stored temporarily along the flood plains during a flood period.

One advantage of channel improvement is, that between storm periods, the water entering the valley is discharged more quickly, with the result that the surface of the water is lowered more rapidly and to a lower level than formerly; and, therefore, at the beginning of the next storm period the channel has a greater storage capacity than before and a longer period will be required for the storm waters to fill the channel even if the increased discharge is not considered. When the increased channel capacity and the increased discharge are considered together, it is evident that the effect of channel improvement is to increase the time that must elapse before the stream leaves its banks due to an excess of inflow.

If the rate of inflow is greater than the discharge of the stream at its mouth or at the lower end of the improvement, and the storm period lasts long enough, flooding will take place eventually, even though it be delayed somewhat. While it is true that the water enters the valley from the nearby slopes and through the tributaries in the same quantities and at the same rate as before, the *distribution of the storage water in the valley is entirely different.* The depth of overflow or the volume of water in storage along the flood plain at the upper end of the stream will be less at all stages of the storm than before the channel was improved, because of the higher rate of discharge; and it may be that flooding at the upper end will be eliminated. The decrease in flooding at the upper end, however, is accompanied by a greater flooding at the lower end. Near the center of the improvement the flooding will be about the same as before for storms of long duration. For while it is true that the new channel at a given point can discharge 50 to 100 per cent more water, it is also true that 50 to 100 per cent more water is being brought to this point from the valley above; and hence the channel is taxed to its full capacity just as it was formerly and has no excess capacity for handling within its banks the water which enters the valley from near-by tributaries. The result would be just as serious a flooding of the middle reaches of the valley as before, although the period of flooding would be shortened.

The lands at the lower end of the improvement are the ones which bear the burden of the unequal distribution of the stored water, unless levees are constructed at the lower end to protect the land and to form a floodway for increasing the discharge at this point. In the assumed problem, the length of the old channel was reduced one-half and the velocity increased 50 per cent, so that water from the upper end would reach the lower end of the improvement in just one-third the former time; and since the water is being discharged at the lower end only about 50 per cent faster than under the old conditions, the result must be a piling up of the water at the lower end and a more serious flood condition.

Effect of Cut-offs on the Regimen of a Stream.—Flowing water by virtue of its velocity tends to erode the material in the bed and banks. This erosive action is resisted by the cohesion of the particles composing the material. When these two forces are equal, no further change will take place in the length or cross-section of a channel. This is the end toward which all streams are working, and all natural channels in alluvial soils are approaching this condition. When the erosive force is greater than the cohesive force, the river tends to balance them by increasing its length, thus reducing the slope and the velocity to a point where the two forces are equal. If the discharge of a stream were constant, a state of equilibrium would be eventually attained; but since the discharge of all streams is a varying quantity, the erosive force also varies since it depends upon the velocity, and the velocity in turn varies as the discharge and the depth of flow. Hence, it is evident that a stream can approach a state of permanent régime but can never attain it. When the erosive force is greater than the cohesion of the material can withstand, cutting of the concave bank occurs at all bends, which increases the sinuosity of the channel. As this process continues, the alternate bends approach more closely to each other until frequently only a narrow neck of land separates two points on a stream which are many miles apart by channel. Occasionally during flood periods, the river overflows its banks and the flood water flows across the point of land to the channel below, and because of its greater slope and velocity, scours a new channel for the stream. These natural cut-offs disturb the regimen of the stream for many miles above and below, and result in a more active erosion within the stretch of increased velocity.

The eroded material is deposited in the channel below the cut-off, and the bed of the stream is raised, which increases flood heights below for the same discharge. The stream will immediately begin to counteract the shortening effect of the cut-off by increasing the extent of existing bends and by starting new ones. Usually the lengthening of the channel occurs chiefly above the cut-off. At the lower end of a cut-off the direction of the current is changed and the river with its increased velocity attacks the banks below at new points, which results in greater or less caving of the banks below, unless they are protected in some way.

Thus it is seen that it is natural for rivers in alluvial soils to be crooked. A cut-off, whether it be natural or artificial, disturbs the balance between flow-strength and bank-strength and affects the regimen of the stream. If left alone, a straightened alluvial stream will in time become as crooked as it was originally. In a small stream the volume of flood discharge is comparatively small, the duration of the flood period is short, and the erosion during a single flood is usually slight, so that by proper attention between floods, the straightened channel can be kept straight; but in a large stream, the damage done during a single flood may be so great that it can neither be prevented nor repaired.

There is undoubtedly a relation between the discharge and slope of a stream and the character of the soil through which it flows. A stream like the Illinois River flowing through glacial drift and carrying comparatively little sediment might be straightened with impunity; but with streams like the Missouri and Red rivers, flowing through sedimentary soils, straightening is a dangerous method of securing flood protection. It is not the increase in the discharge at the lower end of a cut-off which increases flood heights so much as it is the raising of the river bed by the deposition of material scoured from the channel above.

Stream straightening as a means of flood control is generally considered as applicable only to small streams, although there are several eminent authorities who believe that even the lower Mississippi could be successfully straightened.

Cut-offs on the Mississippi River.—There are records of 12 cut-offs on the Mississippi River below Memphis. Some of these were natural and some artificial. The first of these was the Fausse River cut-off below Baton Rouge which occurred in 1776.

Undoubtedly there were many others before this date. The Palmetto Point cut-off just above the mouth of Red River occurred in 1796. Little or nothing is known of the effect of these two cut-offs on flood heights.

The first artificial cut-off was made in 1831 at the mouth of Red River. This cut-off, known as Shreve's cut-off, was made across a neck of land only 800 feet wide and shortened the river 18 miles. A short distance below, Raccourci's cut-off was made in 1848. Here the river was shortened 21 miles by a cut-off about 1 mile in length. These two cut-offs affected the river slopes and gage heights for over 100 miles above and below. At a point about 100 miles above Shreve's cut-off, the maximum stage during the flood of 1851 was the same as that caused by the flood of 1828; but at the head of the cut-off, the stage in 1851 was about 4.6 feet lower than in 1828. Directly below Raccourci's cut-off, the stage in 1851 was about 2 feet higher than in 1828, and at Baton Rouge the stages for the two floods were about the same. This shows the effect of the cut-offs upon flood heights. From a study of these floods, Humphreys and Abbot formed the general conclusion that "a river at the head of a cut-off will be depressed by the whole amount of the elevation at the head of the bend due to the bend's resistance, and by one-half of the fall in a straight part of the river equal in length to the shortening of the river."

Other cut-offs occurred between 1830 and 1863 as follows: Bunch's cut-off in 1830 at a point just below the Arkansas-Louisiana state line; Friar's Point cut-off in 1848; and the Rosendale cut-off in 1863.

In 1866, a cut-off was made at Terrapin Neck about 60 miles above Vicksburg, followed in 1867 by one at Palmyra about 50 miles below Vicksburg. These cut-offs lowered the flood level at Vicksburg about 72 inches, but raised the flood level below the lower cut-off. The caving of the banks below the cut-offs was very disastrous and several large plantations were washed away by the flood waters.

The last cut-off occurred at Coles Point below Waterproof in 1884, and caused great damage below. Because of the destruction to property caused by cut-offs and of the extensive disturbance of the regimen of the river, the Mississippi River Commission adopted the policy of preventing cut-offs by bank revetment at the head of bends, thus confining the river to its present

channel; as a result of this policy, there have been no cut-offs in the last 40 years.

Method of Making Cut-offs.—It is only on very small streams that the cut-off channel is made as large as the natural channel. Ordinarily, a channel with a small bottom width is dredged; for large streams, a greater width may be necessary. The increased velocity through the cut-off section, due to increased slope and the contraction of the channel, will cause rapid erosion, and in a few years the full channel width will be attained in the cut-off section. Most of the eroded material, however, will be deposited in the stretches of lower velocity downstream; or, if the improvement extends to the mouth of the stream, in the backwater at the mouth, thus tending to build a bar which lowers the slope upstream and reduces the effectiveness of the improvement.

Summation.—The foregoing discussion may be summarized in the following general statements:

1. Channel improvement as a *sole* means of flood protection is applicable only to small streams.

2. Only a limited amount of channel straightening is justified on streams of moderate size when levees are used as the main protection against floods.

3. Whether or not the ox-bow bends in large streams should be eliminated by cut-offs is a mooted question, with the weight of opinion against it.

4. The straightening of a portion, only, of a stream, will benefit the land above the improvement and most of the land along the improvement itself, although the benefit decreases from the upper to the lower end. The land at the lower end will generally be damaged instead of benefited.

5. To be effective, channel improvement should begin at the mouth of the stream. Even then the land at the lower end will require protection in the form of levees.

6. The size and relative location of the tributary streams are important factors in determining the extent of channel straightening.

7. Cut-offs on alluvial streams usually result in extensive erosion above the cut-off and in deposition below.

8. Riparian rights also constitute a factor to contend with, especially in the larger navigable streams.

CHAPTER XII

FLOOD PROTECTION BY LEVEES

Levees are small earthen dams placed at varying distances from the banks of a stream to serve as artificial banks during flood periods when the stream gets out of its natural banks, and to protect the major portion of the bottom land from overflow.

Levees are justified and should be used in any valley where the interest on their first cost and the cost of annual maintenance is less than the net annual increase on the returns from the land which is protected by them.

Levees are the oldest known form of flood protection and have been used more extensively for this purpose than any other form of either flood protection or flood prevention. On the lower reaches of long rivers, they afford the only sure means of flood control.

Effect of Levees upon Flood Flows.—The effect of confining the flood waters of a river between levees is: (1) to increase the rate at which a flood wave travels down the stream; (2) to increase the water surface elevation of the river at flood; (3) to increase the maximum discharge at all points downstream; (4) to increase the velocity and the scouring action through the leveed section; and (5) to decrease the surface slope of the stream above the leveed portion.

On an unleveed stream, the advance of a flood wave downstream is delayed by the loss of the water which flows over the banks and collects in depressions in the flood plain. As the depth of water over the flood plain increases, the water will flow down the valley; but for the most part, the velocity is low and the effect is that of storage. Levees, by eliminating valley storage, enable the flood wave to pass downstream undiminished, except for the loss due to the channel storage above the bankful stage provided by the levees. On large streams this channel storage is large. It is estimated that the channel storage on the Mississippi River between the mouths of the Ohio and Red rivers is 30,000,000 acre-feet.[1]

[1] Final Report of the Special Committee on Floods and Flood Prevention, *Trans. Amer. Soc. Civ. Eng.*, vol. 81, p. 1227, 1917.

When the water which formerly spread unhindered over the flood plain is confined between levees, naturally the water must be carried at a higher stage through and below the leveed section. Since the effect of levees is to contract the flood channel, the water above will be held back, thus increasing the flood heights above.

The decrease in the cross-sectional area of the leveed stream, however, is more than offset by the greater velocity caused by the increased slope and hydraulic radius, and the maximum rate of discharge will be greater than before the levees were constructed. As an illustration, the maximum flood flow through the lower reaches of the Sacramento River under natural conditions was about 200,000 cubic feet per second; whereas, the construction of levees will increase the maximum capacity to about 500,000 cubic feet per second.[1]

The tendency to scour is intensified by the construction of levees, because of the greater velocity, and the tendency to deposit sediment is lessened. The scouring is more active on the bed than on the banks, and the general effect is to lower the bed of the stream.

DESIGN OF LEVEES

Flood Discharge.—The first problem to be solved in the design of a levee system is the probable maximum flood discharge which the leveed channel will be called on to carry. This is an extremely difficult question to answer and one which has frequently been answered incorrectly, as evidenced by the fact that on many leveed streams it has been necessary to increase the height of the levees one or more times. There are many factors and contingencies to be considered. First in importance is the run-off factor. At what rate will the upper unleveed reaches of the stream itself, and the tributaries along the leveed portion and along the stream below, discharge their waters at different stages of the maximum storm period? What is the length of storm period to be considered, and what is the total amount of rainfall, and how is this amount distributed over the storm period? What effect will the increased surface slopes below, and the decreased slopes above, have upon the rate of discharge of the tributary streams and upon the time of concentration of tributary flow? What will be the effect when

[1] GRUNSKY, C. E., *Trans. Amer. Soc. Civ. Eng.*, vol. 81, p. 1297.

other portions of the stream itself are leveed and when the tributaries are straightened or leveed? These and other questions must be answered as accurately as the available data will permit.

It is only after repeated losses from floods, or after a specially disastrous flood, that the public is sufficiently aroused to organize for flood protection; hence, there are always data which can be obtained by the engineer as to the discharge and storage at various points on the stream at successive stages of the storm period. These data give the rate of inflow to the river valley and the maximum rate at which the water would have had to be discharged to have prevented flooding. These rates of discharge for the maximum flood of record must then be increased to provide protection against a probable storm of greater magnitude. The method of arriving at an estimate of the probable maximum run-off is discussed in Chapter III. The latter values of discharge must then be modified to take into account the changed condition of flow and of storage caused by the construction of levees. Above the leveed area, the water will be held back because of the contraction of the flood channel. This is true not only of the stream itself, but also of its tributaries; and the increased storage thus provided will elongate the flood wave. Through the leveed portion, the discharge will be greater than when the flood plain was unconfined, and the concentration of tributary flows will be effected more quickly. The discharges thus determined below the mouth of each tributary are used in designing the spacing and height of levees. The fact must not be overlooked, however, that these discharges will be increased when the overflowed areas upstream and along the tributary streams are leveed or otherwise improved, thereby reducing the valley storage and increasing the rate of movement of the flood wave downstream. To provide for this contingency, the floodway between the levees should be increased somewhat and the height of the levees should be less than the maximum, so that they can be safely increased in height whenever the need for greater floodway capacity arises.

Spacing and Height of Levees.—The spacing and height of levees are interdependent. Either the levees can be made low and far apart, or they can be made higher and closer together. The correct location of the levees from an economic standpoint is that which makes the sum of the cost of the levees and the

value of the unreclaimed land in the floodway a minimum. In this connection, the value of the unreclaimed lands should be taken as the difference between the market value of the land, if it were possible to protect it from overflow, and its value to the levee district for floodway purposes.

The spacing and height of the levees are determined by a series of trial solutions. First, a height of levee is assumed. The top of a levee should always be 3 to 4 feet higher than that necessary to carry the estimated discharge. This additional height is called "freeboard," and is provided as a factor of safety. The discharge through and above the channel proper is then computed for the assumed high-water flow; and this discharge subtracted from the maximum discharge gives the discharge which must pass over the floodways between the channel proper and the levees. The latter discharge divided by the computed velocity of flow along the floodway gives the cross-sectional area to be provided by the floodway. This area divided by the depth of flow over the flood plain gives the width of the floodway, exclusive of the channel width. The cost of the levees and the value of the unprotected land in the floodway is then computed. Similar solutions are made for other heights of levees; and, as previously stated, that height of levee (and the corresponding spacing) which gives a minimum value for the sum of the cost of levees and the value of the unprotected land, is the proper height to use. The height thus determined, however, should not be a maximum with respect to the bearing power of the soil upon which the levees are built, or else the height of the levees cannot be raised subsequently to carry the increased discharge caused by future elimination of valley storage above.

In order that the several steps involved in the determination of the correct spacing and height of levees may be better understood, the following example is given.

Illustrative Example.—Given a river channel and flood plain as shown by the cross-section *ABCDEFG* in Fig. 108. The flood discharge to be carried by the leveed stream is 50,000 cubic feet per second; the area of the channel, *CDE*, is 2,050 square feet; the wetted perimeter, *CDE*, is 156 feet; the slope of the stream is 0.0002; n for the channel is assumed to be 0.030, and for the floodway, 0.050. The present value of the land is taken as \$50 an acre and its estimated value when protected by levees as \$150 an acre.

Wetted Perimeter is Average width of Channel.

Assume a trial height of levee of 13 feet, 3 feet of which is freeboard. With the water 10 feet above the bankful stage, the area of the effective channel, *HCDEJ*, will be increased 1,520 square feet, making a total cross-sectional area of 3,570 square feet. The wetted perimeter, *CDE*, is 156 feet, and hence the hydraulic radius will be 22.9 feet. Substituting in Kutter's formula, the mean velocity is found to be 5.82 feet per second; and for this velocity, the discharge would be 20,790 cubic feet per second. Since the flood discharge to be carried is 50,000 cubic feet per second, the floodway must be designed to carry 29,210 cubic feet per second. The velocity of flow along the floodway—for a roughness factor of 0.050, a depth of 10 feet, and a slope of 0.0002—would be 2.19 feet per second. To carry a discharge of 29,210 cubic feet per second at a velocity of 2.19 feet

FIG. 108.—Cross-section for illustrative example.

per second, the area of the floodway must be 13,340 square feet; and since the depth of flow is 10 feet, the width of the floodway must be 1,334 feet, exclusive of the channel proper.

Assume that a levee of the desired crown and side slopes and 13 feet in height will cost $18,000 per mile. Since there is to be a levee on each side of the channel, the total cost of levees per mile of channel would be $36,000. A floodway a mile long and 1,334 feet wide would contain approximately 162 acres. Since the potential value of this land for agricultural purposes is $150 an acre, while its value for floodway purposes is only $50 an acre, the loss is $100 an acre, or $16,200 on a mile of floodway. This loss when added to the cost of levees gives a total of $52,200 a mile.

If a levee height of 14 feet is assumed and similar computations are made, it is found that a floodway width of 1,185 feet, containing 143.6 acres per mile, will be required. Assuming the cost of 1 mile of 14-foot levee to be $20,800, the total cost will be approximately $56,000. Thus there is a difference of $3,800 per mile in favor of the 13-foot levee.

In like manner it is found that a 12-foot levee would necessitate a floodway 1,690 feet in width, and that the total cost would be

$51,500 per mile of channel. Since this is less than the cost of the 13-foot levee, it will be necessary to consider an even lower levee. An 11-foot levee would require a floodway 2,180 feet in width, and involves a total cost of $52,700 per mile. Hence, the 12-foot levee is seen to be the most economical.

Alignment of Levees.—Where there is bottom land to be protected on each side of a stream, it is only just to the land-owners that one-half the floodway should be taken from each side. If the river channel is straight and local conditions permit, the levees can be placed at approximately equal distances from the channel; but usually river channels are winding, and it is not desirable to have the levees follow all the sinuosities of the channel, with the result that the alignment of the floodway is straighter than that of the channel. At no place, however, should the levee be located so close to a bend in the channel that it will be in danger of being undermined by the caving of the river bank, or that its slope will be subject to the erosive action of the higher velocity of the channel current during flood periods. The levees should be as straight as possible and free from sharp bends. All changes in direction should be made by curves of large radius.

The topography of the bottom land will affect the location and alignment of levees. As far as possible, advantage should be taken of any ridges or high ground not only to reduce the cost of the levee, but also to secure a better foundation; likewise, sloughs and other depressions should be avoided.

Freeboard.—The height of a levee above the maximum high-water elevation is known as "freeboard." This extra height is a margin of safety against floods of greater magnitude than that designed for, and affords protection against wave action during storms. Also it tends to prevent the sloughing of the land-side embankment.

The amount of freeboard should be 3 or 4 feet. It is believed that no levee has even broken where the top was 3 feet or more above the water, except through the permeability of the foundation.

Crown.—If ample freeboard and side slopes are provided, the width of the top, or crown, is not of special importance. For example, if a levee has a freeboard of 3 feet, combined side slopes of 6 to 1, and a crown of 6 feet, the thickness of the levee at the water line will be 24 feet; whereas, if the crown were 3 feet instead of 6 feet, the thickness at the water line would be 21 feet, a

reduction of about 12 per cent. If, moreover, in the first instance the side slopes were extended, they would intersect in a vertical distance of 1 foot, making the crown zero and the freeboard 4 feet. Thus, an extra foot of freeboard has the same effect as 5 or 6 feet of extra crown. The width of crown used for levees varies from 3 to 10 feet. Where the levees are low and the flood waters remain at a high stage for a few hours or a day only, a 3-foot crown is sufficient. On the Illinois River, the crowns of the levees vary from 6 to 8 feet. The crown of the standard section of the Mississippi River Commission is 8 feet. Where the top of the levee is to be used for a highway or a railroad, crowns up to 40 feet are used, but these widths are not necessary so far as the safety of the structure is concerned.

A wide crown has the advantage of providing ample space to work in if the water should rise dangerously near the top of the levee and it becomes necessary to raise the height of the levee with bags filled with earth, sometimes from the crown itself.

Side Slopes.—The slopes of the sides of a levee are the most important element in the design, since upon them depend the thickness of the levee at any level and the width of the base. The mass of a levee must be such that planes of saturation will not be formed through it, thereby tending to cause the superincumbent material to slide, and breaks in the levee to occur. The width of the base must be such that any water which may permeate the foundation will not have sufficient force to carry any material with it.

Side slopes are dependent upon the nature of the material of which the levee is composed, the method of construction, the height of the levee, and the length of time that the levee is likely to be subject to the action of flood waters. The steepest slopes which should be used, under the most favorable conditions of construction and of exposure to saturation and erosion, are 2 to 1 on the land side and 2½ to 1 on the river side of the levee. The standard section of the Mississippi River Commission calls for a slope of 3 to 1 on each side. The standard section of the Upper Yazoo District provides a 3 to 1 slope on the river side and a 4 to 1 slope on the land side. Many of the levees on the Upper Mississippi ,which were constructed of sand by hydraulic dredges, have side slopes of 5 to 1.

The height of a levee has a bearing on the side slope which may be used safely on the land side. For levees up to 6 feet in

height, a 2 to 1 slope on the land side may be used; from 6 to 12 feet, a slope of 3 to 1; and for levees over 12 feet, either a 4 to 1 slope should be used or else the toe of the slope should be reinforced with a banquette.

Banquettes.—A banquette is a terrace of earth added to the base of a high levee on the land side to prevent the water from seeping through the base of the levee and causing the levee to slough. The height of the banquette must be such that its surface will be above the plane of saturation, and its width such that the plane of saturation will intersect the natural ground surface within its mass.

The Mississippi River levees have banquettes 20 feet wide for levees from 10 to 13 feet high, 30 feet wide for levees 13 to 16 feet high, and 40 feet wide for more than 16 feet in height. The top of a banquette is usually about 8 feet below the top of the levee, and their surface slopes vary from 1 in 10 to 1 in 20.

The slope of the plane of saturation will vary with the material of which the levee is composed and with the manner in which the levee is constructed. The finer and more compact the material, the steeper will be the slope of the plane of saturation. For the materials and methods of construction used on the Lower Mississippi levees, it has been found from experience that when the plane of saturation extends through the levee and its angle with the horizontal exceeds about 14 degrees,[1] the dry earth in the upper part of the levee will slide on the wet clay below. Hence, to prevent the upper portion of a levee from sliding, the banquette must extend above the point on the land-side slope where a line from the top of the levee on the water side, inclined downward about 14 degrees, intersects that slope.

A banquette not only strengthens the levee itself, but also reinforces the natural ground underneath; and by decreasing the hydrostatic head of the water which seeps under the levee, tends to prevent "blow-outs" or "sand-boils" on the land-side of the levee. A banquette also serves as a place to work in time of danger and supplies material for raising the top of the levee.

Borrow Pits.—The material used in constructing a levee must always be taken from the river side of the levee. A berm of about 40 feet in width should be left between the toe of the levee slope and the edge of the borrow pit. The borrow pit should

[1] Townsend, Col. C. McD., "Levee Construction on Mississippi and Its Effect on River Section," *Eng. Contr.*, Jan. 14, 1920.

be not over 3 feet deep on the side next to the levee and its bottom should slope gently away from the levee to a maximum depth of 8 to 10 feet. The side of the slope of the borrow pit next to the levee should be not steeper than 3 to 1. Borrow pits should not be continuous, but should have "traverses" of undisturbed ground, about 15 feet wide, left at intervals of about 400 feet to prevent the flow of a current along the levee. They should be so constructed that the water will drain out at normal stages of the stream.

Muck Ditches.—Muck ditches are trenches excavated along and under the center of a levee and refilled with the most impervious material at hand to form a cut-off wall under the levee to prevent the water from seeping along the foundation plane of the levee. Such a ditch functions as an exploration trench and gives information as to the subsoil conditions, frequently disclosing conditions which necessitate additional protection against seepage, such as sheet piling. The specifications of the Mississippi River Commission require a muck trench from 5 to 6 feet in width and of the same depth.

The value of a muck ditch is a mooted question among levee engineers. Some maintain that such a ditch is beneficial, while others hold that a muck ditch is not only not a benefit but may be even a detriment. Evidently, the advisability of using a muck ditch depends upon local soil conditions, and it may be used to advantage along certain stretches and omitted at other points where the foundation is satisfactory. Frequently the best material at hand for filling the trench is no better than the material taken out; and in this case, nothing is gained by using the muck ditch. Again, the same amount of money may be spent to better advantage in increasing the levee section. As far as exploration is concerned, shallow borings or test pits would answer the same purpose at a smaller expense.

Levee Grades.—The grade or slope of a levee should be that of the water surface at maximum flood height. This cannot be determined with exactness, since the maximum discharge is never known; but the gage heights which will most likely result at various point from the assumed maximum discharge should be used. On a rising stream the slope of the water surface is greater than the slope of the bed of the stream; while on a falling stream the reverse is true. Hence, in general, the slope of the levees is made approximately the same as that of the bed, unless

unavoidable contractions in the floodway cause a ponding of the water above.

To a certain extent, the 3 or 4 feet allowed for freeboard takes care of the minor errors in the assumptions upon which the design is based, until such time as the grades can be revised as the result of actual flood experience.

Drainage of Levee.—A levee with as steep side slopes as it is practicable to build will withstand the pressure of the water against it, if the levee is dry, that is, if percolation is prevented; but since percolation cannot be prevented, except at large expense, the side slopes and mass of the levee must be such as to withstand the pressure when percolation occurs. A saturated levee is a dangerous one, no matter how large its section may be.

Although the rate of percolation through, and under, a levee can be reduced by an impervious covering on the river side and by sheet piling, eventually the water will penetrate under the levee; and if saturation of the soil is reached, the levee will be endangered. Also, water will percolate more rapidly through damp soil than through dry soil. It is, therefore, important that the levee and its foundation be as dry as proper drainage can make it, both before and during the storm period. To accomplish this end, a line of 4-inch tile is generally laid under the land-side edge of the levee. Sometimes a small, open ditch is used, but a tile drain is preferable. Such drains remove the seepage water as it comes through the levee and tends to prevent—and at all events to delay—the saturation of the levee and foundation, and reduces the hydrostatic pressure of the water in the soil.

Protection from Seepage.—Seepage through a levee can be prevented by covering the riverside slope with concrete, and seepage under a levee can be impeded by driving sheet piling along the toe of the slope, the top of the piling being embedded in the concrete covering the slope. Experimental levees were constructed by the Fourth District, Mississippi River Commission,[1] in 1912 and 1913, to determine the effectiveness of such protection and the cost of the same. The experimental levee was 1,276 feet in length and was built in the rear of the main levee to form a basin. About one-third of the length was built in accordance with the standard specifications of the Commission (including banquette); another third was of standard section without banquette, its river side slope (3 to 1) was covered with

[1] *Eng. News*, Oct. 8, 1914.

a 4-inch concrete slab, and sheet piling was driven 20 feet deep along the toe; the remainder of the experimental levee was constructed with a 5-foot crown and with side slopes of 2 to 1, covered with a 2-inch gunite face protection, and 20-foot wood interlocking sheet piling was driven along the toe. The 4-inch concrete slab protection was composed of 1:3:5 concrete. After several experiments in placing the gunite covering, it was found that the best results were secured by spreading a 2-inch layer of gravel, ranging from ½ to 2 inches in diameter, and spraying it with a very wet gunite, composed of one part cement and three parts sand, forming an impervious mixture 2 to 3 inches thick, at about two-thirds the cost of the 4-inch slab covering.

Between July 18 and 25, the basin was filled with water to an elevation 0.6 foot below the high water of 1912. On July 23, seepage was observed in the standard unprotected levee, and it continued throughout the period of observation. On July 29, seepage was observed at the landside toe of the levee protected by the 4-inch concrete slab. On August 5 and 6, the water in the basin was raised to an elevation 0.4 foot above the high water of 1912. On August 8, the water seeped through the same levee at a point about 15 feet up from the toe. The seepage in both of these places was of small amount, and due probably to the fact that in driving the sheet piling a few buried logs were encountered which prevented a tight cut-off wall at these points. No seepage was observed in the smaller levee faced with gunite, although the water stood against it for more than a month.

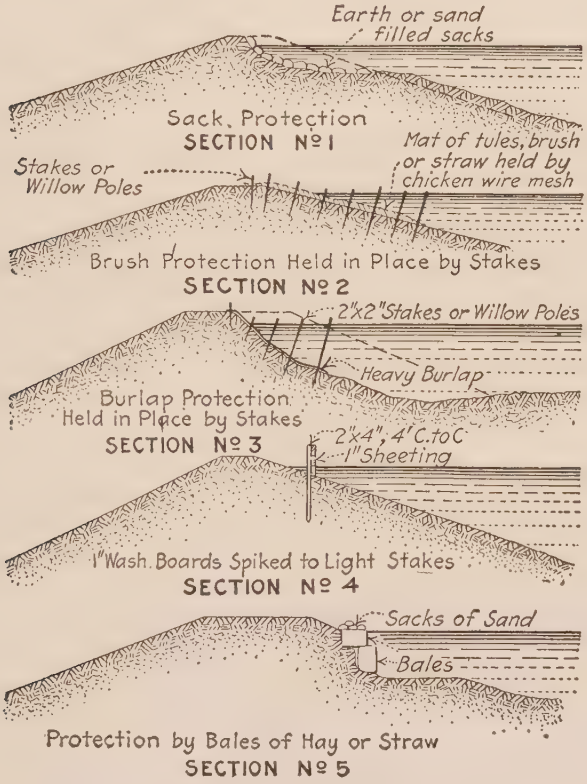
This experiment proved the superiority of levees of small section protected with concrete over the larger levee of earth alone. The cost, however, of levees protected in this way, is such that they cannot be economically used unless the levee is over 16 feet in height, or unless special protection is warranted.

Protection from Erosion by Current.—The riverside slope of a levee must be protected against erosion by the current. Experience on the Mississippi River has shown that the cheapest and most effective protection is secured by a covering of Bermuda grass. Such a covering will resist the action of the current and of minor waves. During the 1913 flood in the Miami Valley, velocities sufficient to scour a macadam roadway did no damage to sod-covered surfaces.

The specifications of the Mississippi River Commission provide that the slopes of levees be sodded with tufts of Bermuda grass

4-inches square placed at intervals of 2 feet. The roots spread quickly and form a dense covering for the levee in about 2 years.

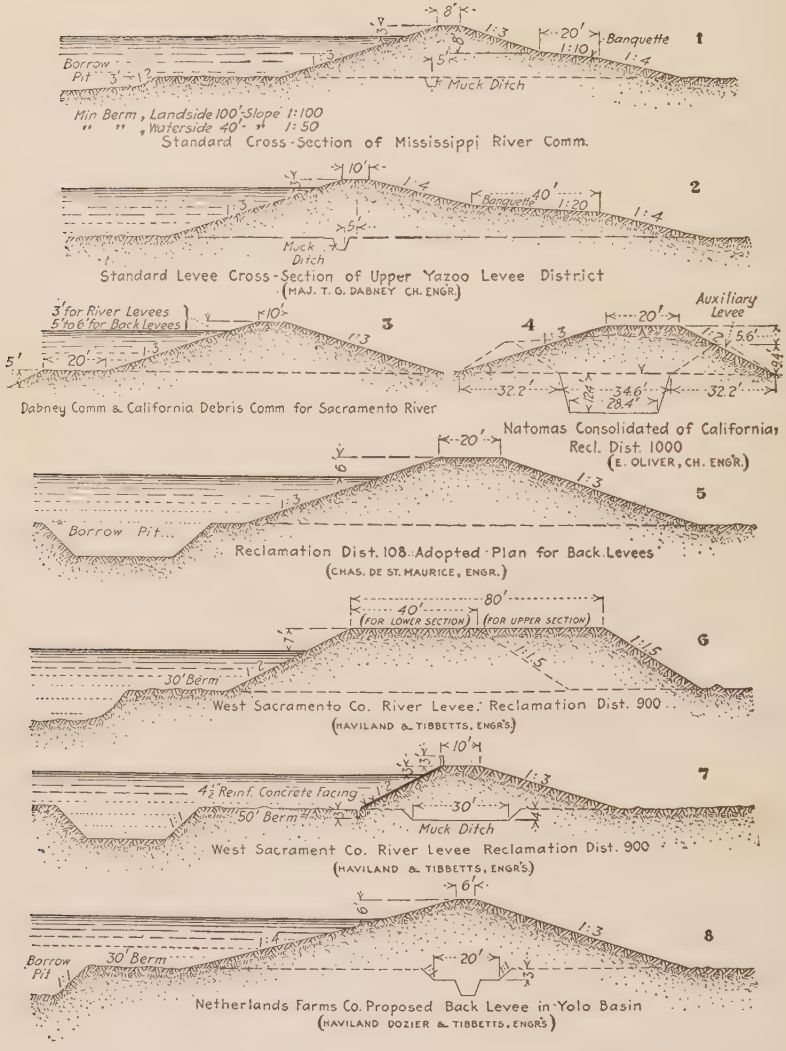
Where the climate is not suitable for the growth of Bermuda grass, other varieties can be used. For clay soils, a mixture of Kentucky blue grass, English rye, and red top is suitable, provided the blue-grass sod itself is not available. The rye and


Earth or sand filled sacks
Sack Protection
SECTION Nº 1
Mat of tules, brush or straw held by chicken wire mesh
Stakes or Willow Poles
Brush Protection Held in Place by Stakes
SECTION Nº 2
2"x2" Stakes or Willow Poles
Heavy Burlap
Burlap Protection Held in Place by Stakes
SECTION Nº 3
2"x4", 4' C. to C.
1" Sheeting
1" Wash Boards Spiked to Light Stakes
SECTION Nº 4
Sacks of Sand
Bales
Protection by Bales of Hay or Straw
SECTION Nº 5

(From Article by F. H. Tibbetts, *Engineering News*, Sept. 4, 1913.)

FIG. 109.—Methods of protecting Sacramento River levees from wave wash.

red top sprout quickly and protect the blue grass which forms the permanent cover. For sandy soils, a quick growing binding grass which will grow with little moisture is necessary. Creeping, bent, and fine-leaved fescue fulfil these requirements. Where the levee is pure sand, it will be difficult to find any vegetal covering which will grow, and in this case the slopes will either have to be made much flatter or else the surface will have to be

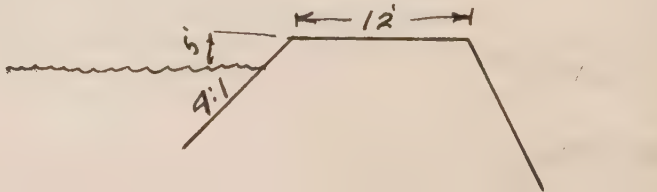


(Taken from "*Engineering News*," September 4, 1913, p. 458.)

FIG. 110.—Comparative cross-sections of Mississippi and Sacramento River levees.

Missippi River Comm. Standard.

Back Slope
5.7:1 at 27'
5.9:1 " 32'
6:1 = " 40'



covered with a revetment of some kind. The Miami Conservancy District experimented with many varieties of vegetation to find some covering suitable for the slopes of their dams, which consist mostly of gravel. They found that sweet clover and alfalfa gave the best results.

Protection against Wave Action.—A grass covering, such as described above, will protect a levee against minor waves; but where the levee is exposed to the full force of waves through long reaches of river, either the sod must be reinforced with a more permanent revetment or flatter slopes must be used. The Upper Yazoo District has used concrete extensively for this purpose. Willows and other small trees have been planted on the berm for this purpose; but trees with heavy roots should not be placed on the slope or near enough to the levee that their roots are likely to penetrate the levee or its foundation and open up passages for the seepage of water. The trunks of trees will not break the force of the waves, but their branches will, to a considerable extent. Coarse gravel spread on the upper portion of the river slope may prove effective in preventing wave wash, if the current is not too strong.

Five methods of protecting the levees along the Sacramento River from wave wash are illustrated in Fig. 109.

Examples of Levee Sections.—Figure 110 shows a number of levee sections along the Mississippi and Sacramento rivers. The section of the levee used by the Natomas Consolidated of California, Reclamation District 1000, is worthy of special note, since it is different from the designs previously discussed. As shown in the figure, a large trench is first excavated along the axis of the levee by a drag-line scraper and the material so excavated is piled up in two ridges as shown by the dotted lines. The trench and the space between the ridges are then filled with sand pumped in by a hydraulic dredge from the river. The sand core is then covered with the more resistant material from the ridges. The sand core is impervious to burrowing animals. Such a levee becomes more impervious with time, since the clay carried by seepage water tends to close the pore spaces in the sand.

CONSTRUCTION OF LEVEES

Materials.—Although the engineer has little, if any, choice as to the material used in a levee, since the material along the line

of the levee must be used in its construction, yet he should thoroughly understand the value of the various types of soils as materials for levee construction. In fact, the intelligent design of a levee will depend upon the material of which it is to be built, and its location may be affected by the proximity of certain materials.

The characteristics of a good soil for levee building are high specific gravity, fineness of particles, cohesion, and irregularity of individual particles. A soil with these characteristics will form a compact mass and will offer great resistance to the percolation of water.

The soil most nearly meeting these specifications is clay containing a small amount of sand. Pure clay shrinks and cracks when it is dry. A small amount of sand in the clay will prevent these occurrences without destroying the toughness of the clay.

Loam ranks next to clay in value. Its particles are fine and pack well. On the other hand, it has little stability when saturated. Levees of loam should be of greater section than those of clay.

Sand has little cohesion and is much inferior to clay and loam as a levee-building material. A levee built of sand must have a large section and a slope on the river side of about 5 to 1, unless the surface is protected from wave wash and seepage. Sand can be used for the core of a levee, if the surface is composed of clay or heavy loam.

Gravel is a valuable material when used in small amounts, mixed with clay. Also coarse gravel makes an efficient surface covering as protection against erosion.

Peat and swamp soils have a very low specific gravity, contain a large amount of organic matter, and, therefore, are not suitable for levee building.

Methods of Construction.—Obviously, no organic matter should be allowed in a levee, since such material will decay and leave openings. All trees, brush, weeds, and grass must be removed from the foundation area. The roots of trees should be grubbed out and the holes filled with good material. Since the topsoil usually contains a large amount of organic matter, it, too, should be removed. The foundation surface should be plowed so as to provide a better bond between the foundation and the levee proper. If a muck ditch is to be used, it is next dug and the

material, if satisfactory, can be used in the base of the levee. Any soft spots disclosed by the trench should be excavated and the trench filled with the best material available, well compacted. The material in the levee should be deposited in layers, starting full out to the slope stakes, and carried up regularly in accordance with the levee section. If there is any difference in the material used, the best should be deposited on the outside of the levee and the inferior material on the inside.

Levees have been constructed by various means from wheelbarrows to power excavators. At present, the revolving drag-line scraper, the cableway drag-line scraper, the grab-bucket excavator, and the hydraulic dredge are used almost exclusively, since these machines can excavate and deposit the material more easily and cheaply than the older types of excavators.

More levees have failed because of improper construction than because of poor materials, and, therefore, the utmost care must be exercised to secure the most compact levee possible with the given materials. The poorer the materials, the greater is the need for careful supervision and construction.

Settlement of Levees.—In staking out a levee, allowance must be made for the settlement which will take place after the levee is completed. The amount of such settlement will depend on the nature of the material, its degree of saturation, the method of construction, and the bearing power of the foundation soil. The more or less saturated material dropping 5 or 10 feet from the excavator bucket to the levee, in masses of 2 to 5 cubic yards, makes a fairly compact embankment; and there will generally be little settlement in the levee itself after it is completed. Most of the settlement of the foundation takes place during construction. The specifications of the Mississippi River Commission require an allowance for settlement of 15 per cent for scraper work, 20 per cent for wagon work, and 25 per cent for drag-line and other machine scrapers. It is believed, however, that under average conditions a well-built levee will not settle as much as this. This specification does not take into account the nature of the material nor its condition of saturation when placed in the levee. The amount of settlement should be left to the judgment of the engineer in the field. Since it is important that the top of the levee, after settlement has taken place, should be at least up to the established grade, it is better to allow too much for settlement than too little.

MAINTENANCE OF LEVEES

Newly built levees require frequent inspection and a considerable amount of general maintenance until the levees cease to settle and their side slope are protected from erosion by a good sod. Heavy rains falling on the soft side slopes may cause gullies to form of sufficient size to require repairs. The tufts of sod may die in places and have to be replaced. Unexpected settlement may occur in places, lowering the crown below the standard grade and requiring additional material to be added to the levee. Or the water may rise against the levee the first season and cause more or less damage to the new levee. The cutting of weeds and repairs of injury by stock are important items. These and other causes necessitate close attention to the levee at first. Burrowing animals are a source of trouble. They cut holes through, and under, a levee, making openings which may prove a source of danger during flood periods. Where apparent, these openings should be filled with good earth during the dry season to prevent excessive seepage and possible breaks during flood periods.

The principal maintenance, however, comes during high water when the safety of the levee is threatened and the crown must be raised to prevent overtopping, or the landside slope reinforced with new material to prevent a break in the levee. Levees may fail in several ways. The usual causes of failure and the methods used to combat them will be discussed separately in the following paragraphs.

Slides.—A common cause of levee failure is the sliding or sloughing of material on the landside slope, due to the fact that, when saturated with seepage water, the earth will not stand at the slope of the dry embankment. Also the finer particles are carried away by infiltration, causing the earth to settle.

Slides do not occur without warning, and if observed in time can generally be prevented. An incipient slide is first indicated by soft spots on the surface of the slope; later, water runs down the slope, carrying particles of earth with it; longitudinal cracks form in the crown or the upper part of the slope; and finally, the earth settles and slides, the extent growing until the pressure of the water causes a crevasse which is usually impossible to control. Slides are usually due to poor construction.

In the first stages, small ditches are cut up and down the slope at intervals of 2 to 6 feet to carry off the seepage water. This

prevents or delays the softening of the surface crust and may ward off the sliding of the bank. If further measures are needed, the

FIG. 111.—Sand bags used to back levee at slough two miles south of Arkansas City.
FIG. 112.—Mattress backing and outside bulkheading at worst point of Fulton slide.

slope is covered with a mat of willows or brush, laid with their trunks parallel to the slope, care being taken not to obstruct the small ditches. The mat tends to prevent the surface crust

from being disintegrated and washed down the slope. To hold the mat in place and to give greater stability to the surface crust, sacks filled with earth are placed at intervals over the mat. If the mat is in danger of sliding, it must be prevented by constructing a bulkhead at the foot of the slope and by weighting the lower portion of the mat with earth-filled sacks.

Another, or additional, method is to dump earth on the water-side slope. This material will tend to fill up the cracks and pores and to reduce the amount of seepage.

Figure 111 shows how a slide was prevented at a point on the Mississippi River levee near Arkansas City during the flood of May, 1922. In this instance, the slough was near the base of the levee and was controlled without much difficulty. Figure 112 shows a more serious situation during the same flood when 400 men labored for 5 days and nights to prevent a break in the levee. About 500,000 sacks[1] were placed on a levee length of about 200 feet. The total settlement in the top of the levee was 9 feet, and a thickness of crown of only 2 feet was left to keep out the river. The crown was reinforced by a bulkhead on the river side as shown in the figure.

Many slides occur along the plane between an old levee and a subsequent landside enlargement. The enlargement of old levees should, therefore, be made by adding material to the river side rather than the land side.

Sand-boils.—Even when a levee is properly constructed and of such mass as to resist the destructive action of percolating water, water may seep through a sand stratum under the levee and break through the ground surface back of the levee in the form of bubbling springs. When these breaks occur, a stream of water bursts through the ground, carrying with it a volume of sand which is distributed around the edge of the hole. Because of the appearance of this occurrence, sand-boils are frequently referred to as "blow-outs." Sand-boils are sometimes as large as 3 or 4 feet in diameter. They usually occur from 40 to 300 feet from the levee and in a few instances as much as 1,000 feet.

If the water flowing from a sand-boil is clear, there is no danger of the levee failing; but when the water is muddy, it means that the levee is being gradually undermined, and that failure of the levee may result from subsidence. A sand-boil

[1] DeBerard, W. W., "Fighting Floods on the Mississippi River above Vicksburg," *Eng. News-Record*, May 18, 1922.

as much as 200 feet from the levee is not a matter of particular concern; but if near the toe of the levee slope, it may prove a very serious matter.

Since sand-boils are caused by the fact that the resistance to percolation offered by the soil particles is less than the pressure of the water due to its head, the method of stopping them is to build embankments of earth-filled sacks around them, thus ponding the water and creating a head on the land side sufficient to stop the flow of silt. If carried high enough to stop the flow of water, other boils might take place outside the encircling embankment. Areas which are subject to sand boils at every

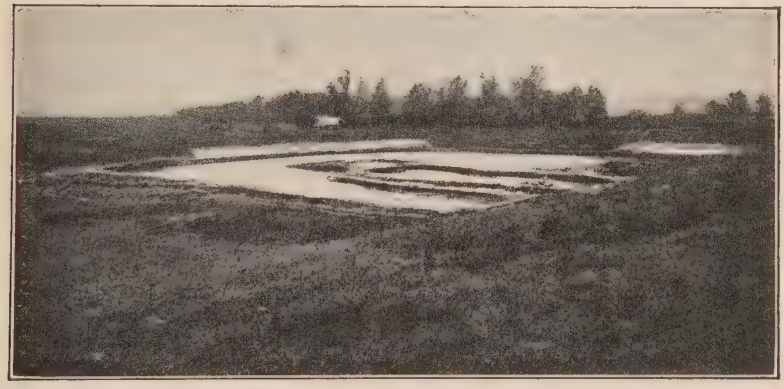


FIG. 113.—Catfish Point loops and sub-levees to hold successive boils.

high water may have permanent sublevees constructed around them.

W. W. DeBerard describes the occurrence and treatment of a sand-boil during the 1922 flood on the Mississippi River as follows:

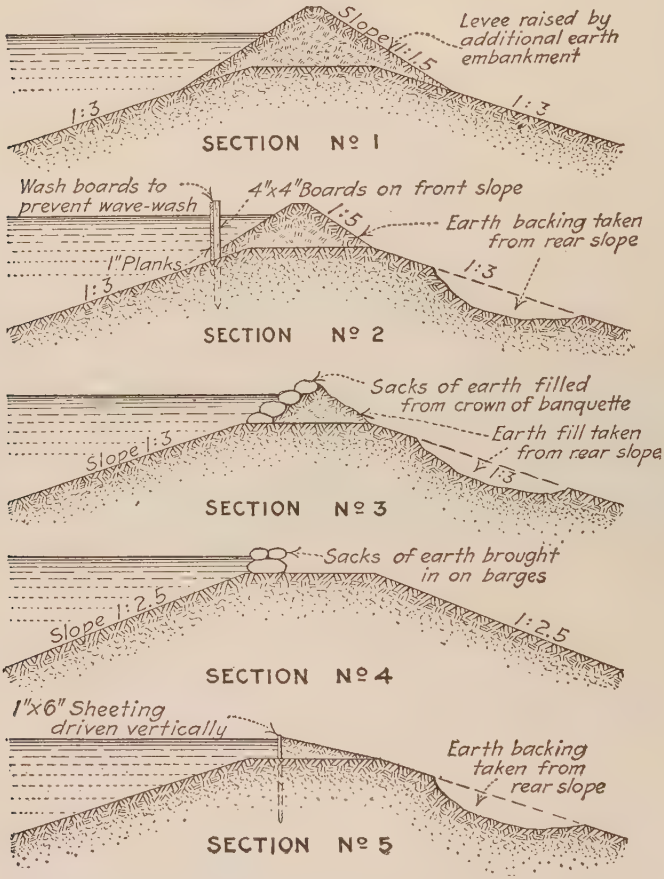
> The most serious boil was at Vaucluse, opposite Greenville, where the crater occurred near the toe of the 40-foot levee crossing an old river bed. It was 3½ feet across and a sack ring 20 feet in diameter was brought up 8 feet before checking it. Before finally coming under control, three concentric loops had to be built, stepping down the pressure three times. The levee settled about 3 inches for a distance of about 250 feet.

Figure 113 shows the sublevees constructed to hold successive sand-boils at Catfish Point during the 1922 flood.

Overtopping.—Insufficient height has been one of the principal causes of levee failure. To prevent the water from running over

the top and washing out a portion of the levee, the grade of the embankment must be raised. Five methods of doing this are illustrated in Fig. 114.

The most common method is that of building up the grade with bags filled with earth. A 100-pound sugar sack is probably



(From Article by F. H. Tibbetts, *Engineering News*, Sept. 4, 1913.)

Fig. 114.—Methods used to prevent overtopping of Sacramento River levees during floods.

the best for this purpose. They should be filled half-full only so that the bags will fit closely against each other. Sand should not be used for filling the sacks, if clay or loam is available. The material is usually taken from the back slope of the levee. The advantage of this method is that it requires only a small

amount of material as compared to the others; but a lack of sacks in sufficient quantity may preclude its use. The method of arranging the sacks is shown in Fig. 115 which is a view of the Mississippi levee at Stopps Landing during the 1922 flood.

An advantage of using the methods shown in Sections 1 and 2 in Fig. 114 is that the work is permanent. Teams and scrapers can be used for hauling the material if the crown of the levee is sufficient; however, in any case, the work will have to be completed with wheelbarrows.

FIG. 115.—Sand-bag topping along levee at Stopps Landing.

The method shown in Section 5 is probably the least expensive and quickest, provided lumber can be quickly and readily obtained. The planking also serves as protection against wave wash.

Wave Wash.—Wherever wave wash is expected to occur, protection against it should be included in the original construction. This may mean an increase in height of levee and some form of permanent revetment. Wave wash, however, may occur where it is not expected, and temporary measures are necessary to prevent the destruction of the levee from this cause. Measures used on the Sacramento River have been referred to and illustrated in Fig. 109.

In addition to these methods, sheets of corrugated iron overlapping longitudinally have been used; also, a floating boom, composed of logs chained together, has been tried.

These methods are expensive, and some cheaper and more permanent method is desirable. A protection of willows or cottonwood trees along the berm is very effective in breaking the force of the waves.

Undermining by River.—It occasionally happens, where a levee is built close to a bend in the river, that the bank at the bend caves to such an extent as to endanger the levee. Such a case occurred near Oldtown, Arkansas, during March, April, and May, 1922. The rate at which the river cut into the levee

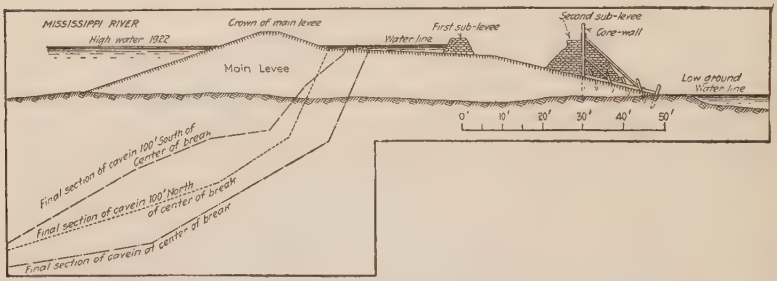


FIG. 116.—Cross-section of levee showing extent of cave-in and emergency sub-levees.

and the extent of the damage are described by L. R. Parmelee in *Engineering News-Record* of June 8, 1922, as follows:

On March 23, with the river reaching flood stage at this point, sections of the revetment failed and the bank had caved angularly to within 39 feet of the levee crown. On March 29, the caving had entered the levee, still angularly, to within 16 feet of the levee crown, and on the night of March 29 it caved into the crown. On the night of March 31, 150 feet of the crown itself caved, leaving a face along the banquette, and on April 2, April 12, April 22, May 5, and May 13, further caves occurred, penetrating the banquette 20 feet in two places and leaving it exposed along a front of 250 feet. The overall width of the caving at this time was 500 feet.

Figures 116 and 117 show the manner in which a break was prevented. An attempt was made to prevent caving of the levee face by covering it with "mattettes," which were constructed of:

. . . 1-inch mesh chicken wire covered with two layers of cotton bagging with a 2- by 4-inch stick laced as a cross-piece at one end, and a 12-inch cylinder, compactly filled with rock and then sealed, at the other. The mattette proper was wrapped tightly around the cylinder, and a ¾-inch tie-line attached to the 2- by 4-inch stick. The roll was then carried to the place at which it was to cover the caving face, eased into the water and held until it was thoroughly saturated and all the air had been excluded from the cylinder, and then turned loose to unroll. The mattettes took the position intended for them very nicely. The mattettes were 5 feet wide and 50 feet long, and had ½-inch steel crossbars every 10 feet of length.

At the same time a sublevee of earth-filled sacks was constructed on the banquette; as a second line of defense, a second sub-

FIG. 117.—General view of break in levee, showing two emergency sub-levees.

levee with a timber corewall was constructed on the toe of the slope. The caving, however, did not extend to the first sublevee, and the second levee was not called into action.

Closing Crevasses.—Once a crevasse has formed in a levee, it widens and deepens rapidly under the action of the swiftly flowing water; and it is very difficult and expensive, if not impossible, to close the opening. The practice is to wait until the flood waters have subsided and then to build a new levee around and back of the old one.

If the levee is comparatively low, the opening not too wide, and the foundation of good material not easily eroded, a crevasse can be closed during high water. The usual method is to drive an open pile dike entirely around the crevasse, to revet the bottom with sacks or other material to prevent underscour, and to fill the dike with brush or with sacks filled with earth. When

sacks are used, the deepest part of the dike should be filled first and the filling carried up horizontally, thus gradually reducing the velocity of flow and making the closure without unnecessary scouring

LEVEES THROUGH CITIES

Where levees are constructed through cities, their design is somewhat different from that of levees across agricultural lands.

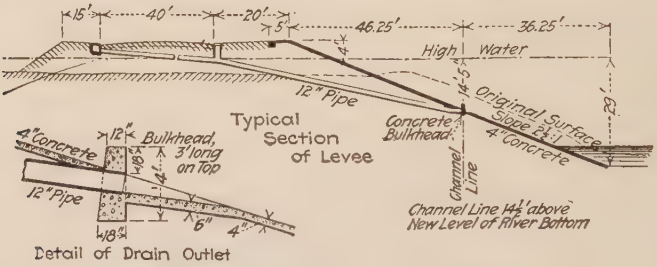


FIG. 118.—Section of White River levee at Indianapolis, Ind.

The property to be protected is much more valuable and, in case of failure, loss of life may be involved. There must, therefore, be no question of failure. If earth is used, the levee section must be such as to give ample protection; and generally the riverside slope is covered with concrete.

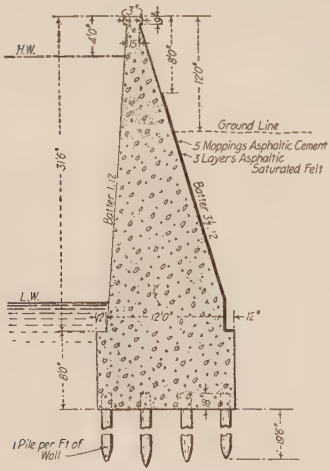


FIG. 119.—Section of river wall at Indianapolis, Ind.

Through the business sections of a city the land is usually too valuable to permit the wide base necessary for an earth levee, and concrete river walls are used instead.

In 1915, the low lands in Indianapolis, Indiana, were protected from the overflow of White River by channel improvement and levees. Figure 118 shows the standard section for the earth levees. The crown was made 75 feet wide to carry a boulevard. The roadway is drained by a 12-inch pipe, discharging midway of the concrete covered slope. North of Washington Street for a distance of 2,300 feet, the

waterfront consists of a concrete retaining wall, a section of which is shown in Fig. 119.

On the Miami River through Dayton and Hamilton, Ohio, both earth levees and concrete retaining walls are used. Through the business section, retaining walls were found to be cheaper than earth levees, because of the valuable property occupied

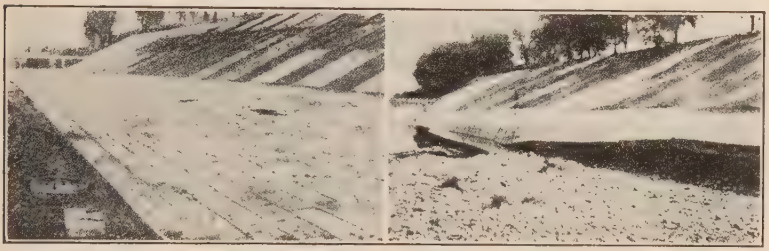


FIG. 120.—Concrete protection of levee of Dayton, Ohio.

FIG. 121.—Effect of April freshet on flexible-slab mat.

by the latter. On the outside of curves and along straight portions where the velocity is extreme, the lower portion of the levee slope is covered with a solid concrete slab, the upper portion being sodded; and from the foot of the levee slope outward, a distance of 10 to 30 feet, a flexible concrete mat is provided as illustrated in Fig. 120. The effect of an April freshet on the flexible-slab mat is shown in Fig. 121.

CHAPTER XIII

FLOOD PREVENTION BY RESERVOIRS

In the two preceding chapters, two methods of flood protection have been discussed. By channel improvement the discharge of a stream is increased by increasing the velocity and possibly the channel cross-section and decreasing the distance the water has to travel to reach the outlet of the watershed. By the construction of levees, the discharge is also increased by increasing the depth of flow, and thereby the velocity, and by providing a floodway outside the channel. Both of these methods give flood control by hastening the flow of the water from the watershed. The method discussed in this chapter gives protection in an entirely different way, namely, by retarding the flow of the water and limiting the flow to the quantity which the channel can safely carry, thereby preventing floods. In using this method, the engineer is simply following the plan used by nature for the regulation of stream flow.

Effect of Natural Reservoirs.—Floods at any point on a stream are caused by the water from above reaching the point in greater quantities than the stream can carry at a bankful stage. Any agency which will delay the flow of water from above will tend to decrease flood heights. Some of the factors which regulate stream flow are flat topography, pervious soils, dense vegetal covering, and lake and swamp areas. Lakes and swamps store the flood waters and discharge them gradually, thus equalizing the flow in the outlet stream. Even if a lake is full at the beginning of a storm period, it still moderates the flow in the stream below, since a considerable increase in the depth of water in the lake is necessary to cause even a small increase in the discharge. The ratio between inflow and outflow will vary directly as the surface area of the lake when full, and inversely as the size of the outlet. The storage in the lake provided by an increment of depth is much greater than the increased discharge caused by the same increment of depth of flow through the outlet. It is only when the depth has been so increased that the flow through the outlet is the same as the inflow that the lake ceases to retard

the flow. This condition is seldom attained—except in small lakes—and even then the discharge below has been reduced by a volume equal to the storage in the lake above the plane of the water surface at the beginning of the storm. It follows, therefore, that a lake, even when full, gives protection against sudden floods and in the case of long-continued floods retards the maximum flow below.

The St. Lawrence River is the best example of a lake-regulated stream. Lakes Superior, Michigan, Huron, and Erie cover about one-third of the drainage area above Niagara Falls. The effect of the storage in these lakes is an almost uniform flow in the St. Lawrence River, the maximum flow being only about 20 per cent greater than the minimum. This difference is negligible when compared with the ratios of maximum and minimum flows of other large rivers. For example, the ratios for the Ohio at Paducah, Kentucky, the Missouri River at its mouth, and the Upper Mississippi River in 1883 were 28.22, 29.00, and 10.29, respectively.[1] The floods which occur in the St. Lawrence Valley are caused not by large volumes of flood discharge, but by ice jams which pond the water and increase gage heights above.

One foot of storage in the four Great Lakes represents a volume of 2,436,000,000,000 cubic feet, an amount greater than the total excess flow of the Mississippi River above the bankful stage at Cairo during great floods.

Although lakes at the head-waters of a stream regulate the flow in the stream below, yet their effect diminishes rapidly with distance downstream; and, if the stream is long, many tributaries enter below, each adding its flow to that in the main stream, and in the lower reaches of the stream the flood discharge may be so great that the amount held in storage by the lake at the upper end is negligible in comparison and entirely inadequate to prevent flooding in the lower portion of the valley.

Under certain conditions, a lake at the head-waters of a stream may prove a disadvantage. The excess water which is held back during the flood period supplies a heavy flow in the stream below for some time; and, if one or more large tributaries crest during this time, the combination of the flows may be greater than it would have been had the lake storage not been available.

[1] CHITTENDEN, COL. H. S., "Reservoir Sites in Wyoming and Colorado," *House Document* 141, Fifty-fifth Congress, Second Session, 1898.

Artificial Reservoirs.—Although many reservoirs have been constructed, their object has been to store water for municipal, irrigation, power, or navigation purposes, and only in a few instances have they been built for flood protection alone. The reason for this is that reservoirs for the first-named purposes are of utilitarian value and return interest on their cost every year; whereas, a reservoir for flood protection proves useful only occasionally, and then its value lies in the prevention of the loss of wealth already acquired rather than in the creation of new wealth.

The cost of reservoirs for flood protection alone is the chief difficulty which stands in the way, although there are also physical difficulties as well. It is possible to construct reservoirs on any stream which will prevent floods below, but in most cases the benefits are entirely incommensurate to the cost. It is obvious that if 1 acre of land must be withdrawn permanently from some productive use to serve as a storage area in order that 3 or 4 acres of bottom land downstream may be protected from overflow once in 5, 10, or 50 years, the investment will not prove a profitable one, especially so when the same protection can be secured at a smaller expenditure. Hence, it is only when the topographic conditions are such that a large storage volume can be secured at a reasonable cost for a dam and for the land within the reservoir basin that this method is economically feasible.

As in the case of lakes, the value of artificial reservoirs in preventing or reducing floods below diminishes rapidly with distance, and if the stream is long, the beneficial effect of a reservoir at the upper end disappears before the bottom land at the lower end of the stream is reached. For example, the six reservoirs constructed at the head-waters of the Mississippi River for navigation purposes, storing 97,000,000,000 cubic feet, reduce the flood flows in the upper part of the valley; but 100 miles below St. Paul their effect upon flood heights is negligible. To be of value, therefore, reservoirs must be comparatively near to the area which is to be protected by them. This distance will depend upon the storage capacity of the reservoir and upon the inflow to the valley below the reservoir. In general, to give effective protection against floods, the capacity of the reservoir should be such as to reduce the flood discharge at the point under consideration by at least one-third.

Applicability of Reservoirs.—From the foregoing discussion it is clear that reservoirs, as a means of flood prevention, are appli-

cable mainly to small watersheds, and that they are especially effective in preventing floods from intense precipitation of the cloud-burst type. The total run-off for small watersheds is such that large reservoirs are not required; and further, the lands to be protected are near enough to the reservoir to receive the protection desired.

For medium-size watersheds of about 5,000 square miles, a single reservoir is seldom sufficient to prevent floods at the lower end of the watershed; but a *system* of reservoirs, one on each of the principal flood producing tributaries, will in many cases prove the solution of the flood problem. The Miami River watershed in Ohio is an example of this type of watershed. The flood protection works recently completed in this valley consist of five retarding basins, one at the head-waters, one on the main stream above Dayton, and one each on the three larger tributaries (see Fig. 124).

For large streams, however, flood protection in the lower reaches cannot be economically secured by the construction of storage reservoirs or retarding basins. For example, it is estimated[1] that to have prevented the Mississippi River from overflowing its banks during the 1912 flood, a reservoir in the vicinity of Cairo, Illinois, would have been required, covering 7,000 square miles to a depth of 15 feet—assuming the reservoir to have been empty when the river reached the bankful stage. Evidently it would be out of the question to use such a large area in this part of the valley for reservoir purposes.

Factors Affecting the Location of Reservoirs.—There are two main requirements which must be met in establishing a reservoir site for flood protection, one physical and the other economic. First, the physical conditions of the watershed must be such that one or more reservoirs can be constructed of sufficient size to store or retard the excess flood waters; and second, the cost of such reservoirs must be reasonable and less than the benefits which will result from their construction.

From a physical standpoint, the topography of a reservoir site must be such that a sufficiently large volume of water can be impounded by the construction of a dam; and the topography and the geologic structure at the lower end must be suitable for the construction of a dam. From an economic standpoint, the dam

[1] Townsend, Col. C. McD., Address before National Drainage Congress, 1913.

must be comparatively short—expecially if it is a high one—and the area within the reservoir must be of little value for other purposes. Because of these requirements, there are comparatively few watersheds where storage reservoirs are practicable as a means of flood prevention.

Since it is the middle and lower reaches of a stream which require flood protection, reservoirs must be located at the headwaters or upon the upper reaches of the stream; or, in the case of large streams, upon the tributaries which contribute to the crest of the flood in the main stream. Then, too, it is upon the tributaries that reservoir sites are most likely to be found which will meet the two requirements mentioned above. For this reason, a system of reservoirs, rather than a single reservoir, is necessary to control flood flows, except on small watersheds where a single reservoir at the upper end may suffice.

Where a number of reservoir sites are available, another factor enters the problem, namely, the effectiveness of each proposed reservoir in reducing the flood crest upon the main stream, and the cost per unit of effectiveness. This factor is brought out more fully in the description of the proposed flood prevention works for the protection of Pittsburgh, Pennsylvania, given in the following section.

Proposed Flood Prevention for Pittsburgh, Pennsylvania.—The city of Pittsburgh, located at the confluence of the Allegheny and Monongahela rivers, has suffered frequently from floods. The direct losses caused by floods in the 10 years from 1898 to 1908 is estimated at $12,000,000. From Mar. 15, 1907, to Mar. 20, 1908, three floods occurred, causing a damage of $6,500,000. Following these floods, the Pittsburgh Flood Commission was formed to study the flood problem and to report upon methods of flood prevention at Pittsburgh.

The engineers reported that storage reservoirs should constitute the major part of the flood protection works. Numerous reservoir sites were available along the many tributaries of the two rivers, and 43 sites were selected and most of them surveyed. The effectiveness of a storage reservoir at each of these sites upon the flood peak at Pittsburgh was then estimated in the following manner.

The magnitude of the flood wave from any tributary of a stream and the length of time required for the crest to reach a given point vary with each stream, because of the variation in

amount and intensity of rainfall in different storms. Hence, the study of the concentration of tributary flow during a single storm is not sufficient. In this instance, therefore, the 11 storms between 1898 and 1908 were studied separately. As in all studies of this kind, the lack of sufficient detailed rainfall data constituted a serious difficulty, as did also the lack of definite information on the time of concentration of the flood waters.

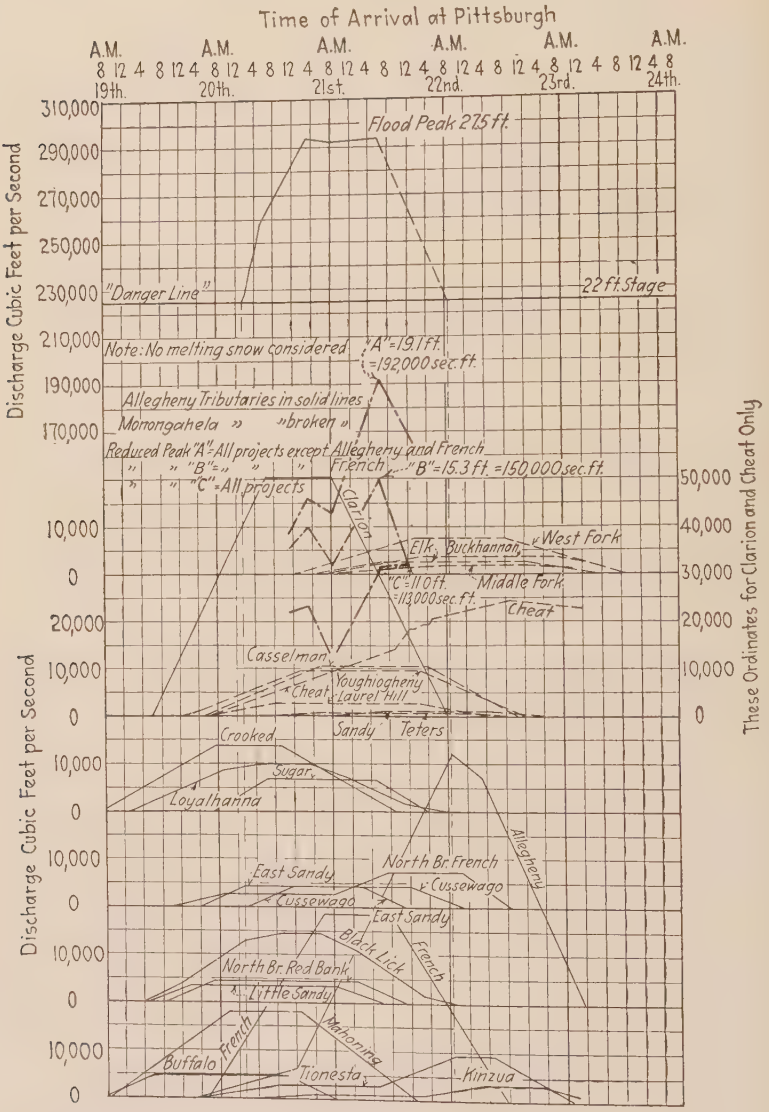
Assumptions as to Rate of Rainfall and Time of Collection.—The following assumptions were made:

1. That on the day of the greatest rainfall the rain fell throughout the 24 hours at a uniform rate. The hourly rate of rainfall for that day, therefore, would be the total rainfall divided by 24. If either the day before or the day after, or both, had rainfall, it was assumed that this rain fell at the same rate as on the principal day. The amount of rainfall on these days, therefore, was divided by the hourly rate obtained for the principal day and the number of hours of rainfall during these 2 days, obtained in this manner. If there were more than 3 consecutive days of rainfall, the first and last days were assumed to have their rainfall at the same rate as the following and preceding days, respectively. In this way a time of starting and stopping of the rain was determined for purposes of calculation.

2. That after the beginning of the rainfall it takes 1 hour for the water running off to get to the small tributaries; 2 hours for it to travel through these small tributaries; and a certain number of hours, varying with the particular stream, for the water to travel the length of the stream to the dam nearest the mouth. This time of travel down to the dam has been estimated in each case from a study of the average slope, the shape and size of the drainage basin, the distance to be traveled, and, where records are available, from an inspection of the velocities during discharge measurements.

The time of movement down the main stream has been computed by means of the Chezy formula from the best available information.

Flood-flow Diagrams.—The maximum rate of discharge during each flood for each tributary upon which it was proposed to construct a reservoir was computed by multiplying 75 per cent of the hourly rainfall as recorded at the nearest rainfall station, expressed in cubic feet per second per square mile, by the drainage area above the proposed dam nearest the mouth of the tributary. A flood-flow diagram for each tributary for each storm was then plotted, using the discharges as ordinates and hours as abscissas. In plotting, the discharge at the beginning of the rain was taken as zero, and it was considered to increase uniformly



(Plate 63, Report Pittsburgh Flood Commission.)

FIG. 122.—Flood-flow diagram for storm of Apr. 21, 1901, at Pittsburgh, Pa.

to its maximum value in the number of hours estimated as the time of collection. The discharge was assumed to remain at this rate until the storm stopped, after which it decreased gradually to zero in the same time as that required for its rise.

The flood-flow diagrams for the various tributaries for the same storm and the hydrographs of the flood peak at Pittsburgh were then plotted on the same sheet, as illustrated in Fig. 122, which is the sheet for the storm of Apr. 21, 1901. It will be observed that the flood-flow diagrams do not start at the same time, but that the zero discharge of each diagram is plotted at an abscissa equal to the time required for the water from that tributary to flow to Pittsburgh.

Per Cent of Effectiveness.—As the "danger line" at Pittsburgh is the 22-foot stage, only the flood peak above that stage is shown. The total discharge contributed by each tributary to the flood peak at Pittsburgh can be determined by measuring the volume of flow represented by that portion of the area of each flood-flow diagram which is directly below that portion of the Pittsburgh flood peak above the danger line. This volume of flow was called the *effective volume of discharge.* The ratio of the effective volume of discharge for a given tributary to the sum of the effective volumes of discharges for all the tributaries considered was called the *per cent of effectiveness.* The per cent of effectiveness for each tributary was then multiplied by the total reduction in the peak flow at Pittsburgh, which would have occurred if there had been a reservoir on each of the tributaries at the proposed reservoir site; and the products represented the reduction of the peak flow at Pittsburgh which would have resulted from the storage of the flood flow in each reservoir, respectively. The total reduction in the peak flow at Pittsburgh is shown by the heavy dot-and-dash line shown at *C* in Fig. 122.

Effectiveness of Reservoirs.—The results obtained from an analysis of the 11 flood periods considered are shown in Table XLIX, which shows for each tributary the amount in feet by which a reservoir on that tributary would have lowered the flood stage at Pittsburgh for each of the 11 storms, assuming the reservoir of sufficient capacity to store the entire flow.

A study of Table XLIX shows the effect of the variation in the distribution of rainfall over the entire watershed during different storms upon the discharge contributed by the several tributaries to the flood peak at Pittsburgh. For example,

TABLE XLIX.[1]—RELATIVE VALUE OF EACH RESERVOIR PROJECT IN REDUCING GAGE HEIGHTS AT PITTSBURGH, PENNSYLVANIA

Project		Reduction in gage height in feet											
Reference No.	Name	Mar. 20, 1908	Feb. 16, 1908	Mar. 15, 1907	Mar. 22, 1905	Mar. 4, 1904	Jan. 23, 1904	Mar. 1, 1903	Mar. 1, 1902	Apr. 21, 1901	Nov. 27, 1900	Mar. 24, 1898	Average of 11 storms
	Maximum stage	27.3	30.7	35.5	29.0	26.9	30.0	28.9	32.4	27.5	27.7	28.9	
1	Allegheny No. 2	2.30	2.31	0.36	1.30	0.96	0.85	0.84	0.68	1.32	3.15	1.15	1.37
2	French	1.23	1.35	0.98	0.54	0.89	1.14	1.32	1.14	2.00	1.36	1.28	1.16
3	Clarion No. 1	0.93	0.93	0.46	1.35	0.94	0.78	1.24	1.35	1.50	0.55	0.75	1.00
4	Clarion No. 3	0.92	0.90	0.42	1.30	0.91	0.80	1.20	1.30	1.43	0.48	0.70	0.94
5	Cheat No. 2	0.28	1.13	0.72	0.49	0.32	0.65	0.97	1.24	0.52	2.16	0.88	0.82
6	Allegheny No. 1	1.37	1.38	0.22	0.76	0.58	0.50	0.49	0.40	0.78	1.88	0.66	0.81
7	Allegheny No. 3	1.26	1.24	0.20	0.70	0.52	0.46	0.47	0.38	0.72	1.70	0.64	0.70
8	Tionesta	1.04	1.23	0.66	0.52	0.57	0.96	0.63	0.67	0.38	0.73	0.53	0.70
9	Black Lick	0.87	0.66	0.46	0.40	0.65	0.51	0.22	0.55	0.88	0.95	0.60	0.70
10	Cheat No. 1	0.20	0.86	0.58	0.42	0.24	0.55	0.76	0.98	0.42	1.70	0.70	0.68
11	West Fork	0.13	0.65	0.42	0.30	0.08	0.44	0.49	0.53	0.35	0.47	0.08	0.36
12	Mahoning No. 2	0.40	0.48	0.31	0.18	0.48	0.30	0.12	0.20	0.58	0.35	0.54	0.31
13	Crooked	0.58	0.44	0.40	0.10	0.42	0.25	0.08	0.17	0.60	0.17	0.58	0.31
14	Clarion No. 4	0.30	0.29	0.12	0.42	0.30	0.26	0.40	0.40	0.45	0.15	0.22	0.30
15	North Br. French	0.29	0.29	0.26	0.09	0.09	0.58	0.27	0.28	0.50	0.50	0.18	0.27
16	Loyalhanna	0.17	0.27	0.41	0.10	0.29	0.01	0.06	0.17	0.60	0.40	0.48	0.26
17	Youghiogheny No. 2	0.41	0.29	0.29	0.07	0.09	0.20	0.22	0.15	0.28	0.40	0.20	0.20
18	Kinzua	0.39	0.27	0.21	0.22	0.08	0.24	0.24	0.21	0.07	0.38	0.13	0.19
19	Laural Hill	0.35	0.21	0.30	0.10	0.15	0.18	0.07	0.23	0.26	0.31	0.15	0.19
20	Shavers Fork No. 2	0.06	0.28	0.17	0.12	0.07	0.15	0.23	0.30	0.10	0.52	0.20	0.19
21	Mahoning No. 1	0.24	0.29	0.18	0.09	0.28	0.18	0.06	0.12	0.38	0.17	0.32	0.19
22	Buckhannon	0.02	0.23	0.31	0.18	0.01	0.13	0.26	0.37	0.13	0.14	0.05	0.18
23	N. Br. Red Bank	0.14	0.22	0.10	0.05	0.30	0.20	0.08	0.14	0.31	0.24	0.24	0.18

24	Youghiogheny No. 4	0.30	0.21	0.21	0.05	0.07	0.15	0.18	0.13	0.20	0.33	0.15	0.17
25	Cussewago	0.12	0.09	0.11	0.03	0.13	0.25	0.15	0.13	0.34	0.20	0.09	0.12
26	Little Sandy	0.14	0.20	0.08	0.05	0.19	0.12	0.05	0.08	0.16	0.10	0.18	0.12
27	Youghiogheny No. 5	0.23	0.15	0.14	0.04	0.05	0.10	0.12	0.09	0.18	0.20	0.09	0.10
28	Elk	0.02	0.18	0.16	0.09	0.02	0.13	0.15	0.14	0.10	0.13	0.03	0.10
29	Sandy	0.07	0.14	0.19	0.06	0.01	0.10	0.10	0.10	0.02	0.07	0.06	0.10
30	Youghiogheny No. 1	0.15	0.14	0.10	0.03	0.04	0.08	0.09	0.07	0.10	0.15	0.08	0.09
31	Casselman No. 5	0.13	0.09	0.06	0.04	0.04	0.08	0.12	0.07	0.10	0.11	0.09	0.09
32	Buffalo	0.15	0.08	0.08	0.04	0.13	0.01	0.02	0.03	0.10	0.03	0.20	0.06
33	Middle Fork No. 2	0.02	0.10	0.16	0.09	0.01	0.03	0.08	0.13	0.04	0.03	0.03	0.06
34	Youghiogheny No. 3	0.13	0.09	0.09	0.03	0.02	0.08	0.06	0.05	0.09	0.13	0.06	0.06
35	Casselman No. 3	0.10	0.07	0.03	0.04	0.04	0.06	0.08	0.05	0.09	0.09	0.06	0.06
36	East Sandy No. 1	0.06	0.07	0.03	0.02	0.12	0.05	0.02	0.01	0.09	0.09	0.05	0.06
37	Shavers Fork No. 1	0.02	0.08	0.05	0.03	0.02	0.05	0.06	0.10	0.03	0.15	0.06	0.06
38	Teters	0.03	0.10	0.09	0.04	0.01	0.06	0.05	0.06	0.05	0.10	0.03	0.05
39	Casselman No. 2	0.07	0.04	0.02	0.03	0.02	0.04	0.05	0.02	0.05	0.06	0.04	0.05
40	Casselman No. 4	0.08	0.06	0.03	0.03	0.02	0.05	0.06	0.05	0.07	0.06	0.05	0.05
41	East Sandy No. 2	0.03	0.05	0.02	0.01	0.08	0.03	0.01	0.02	0.06	0.06	0.05	0.04
42	Casselman No. 1	0.05	0.03	0.02	0.01	0.01	0.02	0.05	0.02	0.01	0.04	0.02	0.03
43	Middle Fork No. 1	0.02	0.03	0.05	0.02	0.01	0.01	0.03	0.05	0.01	0.02	0.01	0.03

[1] Compiled from data on Plate 66, Report of Pittsburgh Flood Commission.

the Allegheny No. 2 watershed contributed the largest discharge during the flood of Mar. 20, 1908, but during the flood of Mar. 15, 1907, it ranked eleventh in effectiveness, and during the flood of Mar. 1, 1902, it ranked sixth.

The last column in the table gives the average reduction for the 11 storms, and the projects are arranged in the order of importance indicated by the data in that column.

Cost per Unit of Effectiveness.—In comparing the effectiveness of reservoirs, not only the amount of the reduction in the peak flow must be considered, but also the cost of the reduction, that is, the importance of a reservoir is indicated by the cost per unit of effectiveness. Table L gives the rank of each project during each of the 11 storms on the basis of cost per unit of effectiveness. Thus, in the storm of Mar. 20, 1908, project No. 27 ranked first, project No. 9 second, etc. The last column gives the rank of the projects when the average cost per unit of effectiveness for the 11 storms is considered.

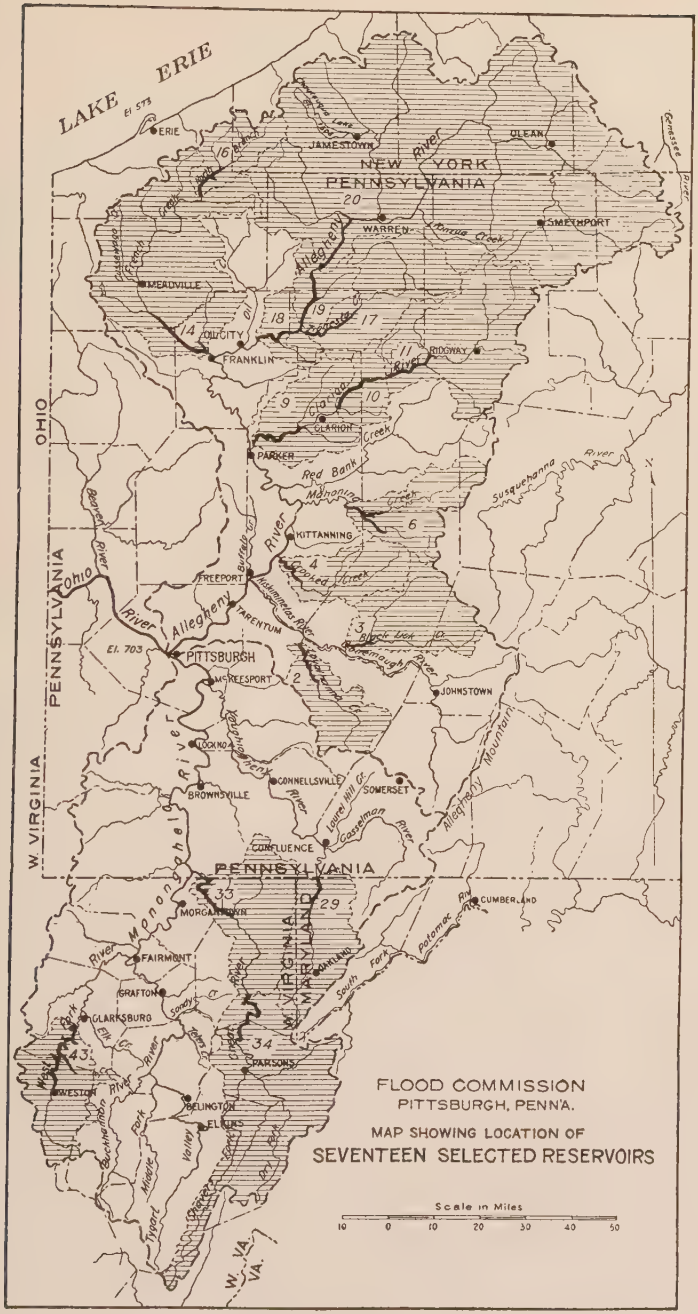
Result of Study.—As a result of an analysis of the data in these two Tables XLIX and L, a number of projects were eliminated as being negligible in reducing the peak flow at Pittsburgh. First, 15 projects were eliminated and new tables prepared, since the elimination changed the per cent of effectiveness of the remaining projects. A study of these tables indicated that a further elimination could be made, and finally 17 projects were chosen and the construction of reservoirs recommended. Table LI gives these 17 projects in the order of their importance, based on the per cent of effectiveness in reducing the flood peak at Pittsburgh.

A map of the watershed above Pittsburgh, showing the location of the 17 recommended reservoirs, is given in Fig. 123.

In some cases it was found that the capacities of the reservoirs were not sufficient to store the effective volume of discharge, and in these instances the effective capacities only were considered in the final analysis.

The total cost of the 17 projects as estimated by the Flood Commission was $21,672,100, but a Board of United States Army Engineers in reviewing the report increased this estimate to $34,000,000.

Effect of Reservoirs on Lower Ohio.—The combined storage capacity of the 17 reservoirs, as shown in Table LI, is 59,481,000,000 cubic feet, and the watershed area controlled by the reservoirs is 10,152 square miles. Although the storage of 59,481,000,000



(Plate 70, Report Pittsburgh Flood Commission.)

FIG. 123.—Location of 17 reservoirs recommended by Pittsburgh Flood Commission.

TABLE L.[1]—RELATIVE VALUE OF RESERVOIR PROJECTS BASED ON COST PER UNIT OF EFFECTIVENESS

Order of value	Project numbers from Table XLIX											
	Mar. 20, 1908	Feb. 16, 1908	Mar. 15, 1907	Mar. 22, 1905	Mar. 4, 1904	Jan. 23, 1904	Mar. 1, 1903	Mar. 1, 1902	Apr. 21, 1901	Nov. 27, 1900	Mar. 24, 1898	Average of 11 storms
1	27	22	22	4	4	15	22	22	4	6	9	9
2	9	27	33	3	9	4	4	4	9	27	27	4
3	7	6	9	14	14	27	3	3	3	10	4	22
4	4	9	24	22	23	8	27	14	27	9	13	3
5	24	7	28	6	6	9	14	10	14	7	3	14
6	8	8	11	9	13	3	10	9	2	5	6	6
7	1	4	8	7	8	22	11	5	13	1	10	7
8	3	1	10	33	12	14	5	33	22	20	2	10
9	14	11	2	1	27	11	2	27	15	24	5	8
10	13	3	5	8	7	2	33	11	6	22	14	2
11	2	10	13	11	3	28	8	8	23	15	12	5
12	15	5	4	27	36	10	24	2	12	8	23	1
13	35	14	15	10	2	23	28	20	24	2	7	11
14	17	33	3	28	1	6	6	28	7	11	32	33
15	42	24	16	5	21	24	15	15	25	4	16	13
16	18	28	29	2	26	5	20	6	16	23	24	24
17	12	2	14	18	41	25	35	24	1	28	1	15
18	31	13	12	24	32	7	7	23	11	3	8	23
19	39	23	20	20	16	1	9	7	21	17	21	28
20	32	12	19	35	25	12	31	1	28	14	20	20
21	30	20	38	12	24	13	42	13	35	18	26	12
22	40	15	17	23	10	20	39	35	10	35	22	35
23	23	21	21	31	5	35	1	18	5	16	15	21
24	19	26	30	42	15	18	40	19	26	31	33	31
25	21	17	23	29	35	33	25	29	39	42	35	17
26	26	35	25	39	31	31	18	25	8	39	31	42
27	36	38	6	15	19	42	17	31	36	30	42	36
28	34	18	18	13	42	39	29	42	33	12	36	39
29	41	29	32	38	20	26	38	12	42	36	39	16
30	25	42	35	26	39	21	23	39	17	25	17	25
31	11	31	34	40	40	17	30	36	40	40	40	18
32	10	39	31	32	11	40	12	40	31	38	41	26
33	5	36	42	21	17	29	34	17	41	19	30	40
34	16	30	7	19	28	36	37	38	32	37	25	41
35	29	40	39	16	30	30	13	41	30	41	18	29
36	6	16	36	17	18	19	26	26	19	21	29	19
37	28	41	40	36	34	38	21	21	20	34	19	30
38	20	19	1	30	38	41	19	37	34	33	34	38
39	38	34	41	41	37	34	16	16	38	13	28	32
40	22	32	26	37	22	37	43	30	18	26	11	34
41	33	25	27	34	33	16	32	34	29	29	37	37
42	37	37	43	25	43	43	36	32	37	32	38	27
43	43	43	37	43	29	32	41	43	43	43	43	43

[1] Compiled from data on Plate 67, Report Pittsburgh Flood Commission.

cubic feet would prevent dangerous flood heights at Pittsburgh, yet this storage is altogether insufficient to prevent floods on the Lower Ohio River. During the flood of March-April, 1913, over three times this volume of storage would have been required to prevent the river from rising above the danger stage at Louisville, Kentucky; and at Cairo, Illinois, about eleven times the Pittsburgh storage would be necessary, representing an expenditure of about $375,000,000.

TABLE LI.[1]—COST AND EFFECTIVENESS OF 17 RESERVOIR PROJECTS PROPOSED FOR FLOOD PREVENTION AT PITTSBURGH, PENNSYLVANIA (1910)

Project	Average reduction gage height Pittsburgh, feet	Area watershed controlled, square miles	Height of dam, feet	Length of crest, feet	Capacity, million cubic feet	Total cost	Cost per million cubic feet
Allegheny No. 2	1.37	.....	66	1,670	4,877.9	$ 3,057,000	$627
French	1.34	1,009	75	1,550	3,323.1	2,388,100	719
Cheat No. 2	1.12	.....	136	1,130	7,294.1	1,721,900	236
Clarion No. 1	1.00	1,212	142	880	5,067.1	1,309,900	258
Clarion No. 3	0.97	.....	128	800	4,886.6	1,028,100	210
Allegheny No. 3	0.95	.....	54	1,415	2,663.6	1,386,000	520
Allegheny No. 1	0.81	3,795	63	810	2,876.3	1,219,900	424
Tionesta	0.71	477	103	800	3,629.6	1,362,600	375
Black Lick	0.69	414	63	1,330	1,454.7	720,500	495
Cheat No. 1	0.68	1,399	113	1,040	5,737.4	1,287,900	224
Youghiogheny No. 2	0.59	394	76	1,270	1,547.1	999,300	646
West Fork	0.35	366	66	750	2,724.3	811,200	298
Mahoning No. 2	0.33	335	143	740	2,367.8	1,089,400	460
Crooked	0.32	287	94	1,100	3,255.7	893,700	274
Clarion No. 4	0.30	.....	70	880	1,537.9	451,800	294
North Br. French	0.27	217	67	1,015	2,125.7	722,800	340
Loyalhanna	0.26	277	112	1,370	4,122.5	1,222,000	297
Grand totals and averages	12.06	10,182	93	1,091	59,481.4	$21,672,100	$364

[1] Reproduced, with modifications, from Plate 69, Report of Pittsburgh Flood Commission.

Reservoirs for Flood Prevention and Other Uses.—From the above discussion it is clear that the chief obstacle to the construction of reservoirs for flood prevention alone is the cost. For small watersheds, such projects are economically feasible; but for large watersheds, the enormous cost prohibits their construction.

To lower the cost of flood prevention, it has frequently been proposed to use the reservoirs for other purposes as well as for flood control. The requirements for other uses, however, are

more or less antagonistic to those of flood control. In order that a reservoir may prevent floods, it must be empty at the beginning of the storm period, and after each storm it must be emptied as rapidly as is consistent with the capacity of the outlet channel, so as to be ready for the next storm which may closely follow the first one; whereas, to be of value for water power, navigation, irrigation, or municipal purposes, the storm run-off must be conserved and used gradually or intermittently. For water power and municipal uses, a more or less uniform daily flow is required; for irrigation, the water is withdrawn from the reservoir only during the growing season; and for navigation, the water is used only in the summer months to increase the low-water discharge. A reservoir with a capacity just sufficient to store the necessary volume of flood water, therefore, cannot be used for any other purpose. Where it is practicable, however, to construct a reservoir of much greater capacity than that needed for flood control, the additional capacity can be used for other purposes; but the storage required for flood control must always be available for that purpose; and it is here that the danger lies, since it is difficult to prevent the commercial users of the reservoir from encroaching upon the storage capacity reserved for flood control.

Many reservoirs have been constructed in Europe in part for flood control, and in part for water supply or for water-power development. Most of these reservoirs are small, however, storing less than 1,000,000,000 cubic feet of water. The Pinay Reservoir on the Loire River in France was constructed in 1711 and is of the detention basin type. The capacity of this reservoir is about 3,500,000,000 cubic feet. The Mauer Reservoir on the Bober River in Germany, built in 1905 and storing 1,765,000,000 cubic feet, was designed for flood control and water power, 60 per cent being reserved for the former purpose. The largest reservoir of this kind in Austria is the Königreiche-Walde Reservoir on the Elbe River, which has a capacity of 320,900,000 cubic feet, 83 per cent of which is reserved for flood control.

Of the large reservoirs constructed in the United States by the Reclamation Service, only one—the Elephant Butte Reservoir in New Mexico—was designed with flood control in mind. Storage in this reservoir is primarily for irrigation purposes, but 400,000 acre-feet are reserved for flood control. All the reservoirs constructed by the Federal Government, however,

have in a large measure regulated the flow of the streams on which they are constructed.

The proposed reservoir at Boulder Canyon on the Colorado River will serve three purposes—irrigation, flood control, and water power. At this point, it is possible to construct a reservoir of 30,000,000 acre-feet capacity, and if 5,000,000 to 6,000,000 acre-feet are reserved for flood control, the flow of the Colorado River can be confined to less than 25 per cent of its maximum flood discharge.[1]

Detention Reservoirs.—There are two classifications of reservoirs, the storage reservoir and the detention reservoir. A storage reservoir has manually operated gates in the base of the dam so that all the flood water can be stored and released as desired. A detention reservoir, or retarding basin as it is commonly called, has one or more openings in the base of the dam which are open at all times, and retains only the water in excess of that which the openings can discharge and the outlet stream can safely carry.

The chief advantage of the storage reservoir is that near the peak of the flood the gates can be closed and the discharge from the reservoir completely shut off. The principal disadvantage is that the gates might not be opened and closed at the proper time; and in case of emergency might refuse to operate. Those in charge of the dam might fail to empty the reservoir soon enough after one flood and a second flood, closely following the first, would find the reservoir full or partly so.

The main advantage of the detention reservoir is that the discharge from it is free from human control and that it is certain to fulfil the function for which it was designed. The openings in the base of the dam can be made of such size that the discharge from them can never exceed a volume which the stream below can safely handle. The only disadvantage is that the water is stored only temporarily and can be used for no commercial purpose. The detention reservoir is strictly a flood control reservoir.

Detention basins have been used in Europe for several centuries. The reservoirs on the Loire River at Pinay and LaRoche, France, constructed in 1711, are of this type.

[1] Davis, Arthur P., "Flood Problems in the Arid Regions," Director, United States Reclamation Service, *Trans. Amer. Soc. Civ. Eng.*, p. 1487, 1922.

The best example of the use of detention reservoirs for flood control is the work of the Miami Conservancy District in Ohio, which is described in the following section.

Detention Reservoirs of the Miami Conservancy District.[1]—Following the disastrous flood in the Miami River Valley in

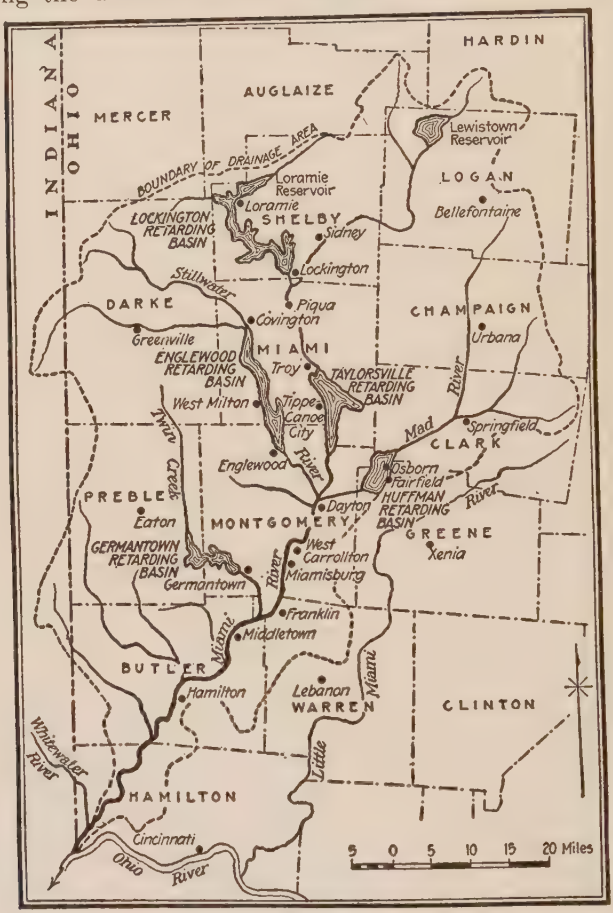


FIG. 124.—Location of retarding basins in Miami River valley.

1913, the Conservancy Act of Ohio was passed and the Miami Conservancy District organized. The chief flood prevention works of this district consist of five detention reservoirs, located as shown on the map in Fig. 124. Data concerning these reser-

[1] The information in this section has been compiled from *Technical Reports* Parts 3 and 7 of the Miami Conservancy District.

voirs are given in Table LII. The combined capacity of the five reservoirs is 1.5 times the valley storage above Hamilton during the 1913 flood.

The bodies of all the five dams are of earth, because the solid rock formations are so deep under the surface that the cost of

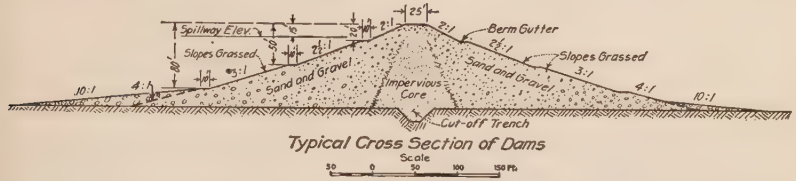


FIG. 125.—Cross-section through hydraulic fill earth dam forming retarding basin.

concrete dams was prohibitive. As may be seen from Fig. 125, which shows a typical cross-section, the dams are exceptionally massive to insure absolute safety to the densely populated valleys below. The dams were constructed by the hydraulic method

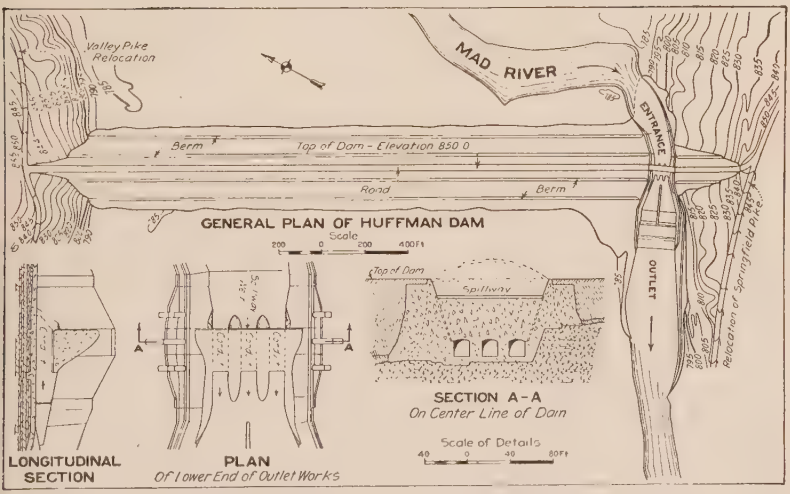


FIG. 126.—General layout of Huffman dam.

in such a manner as to place the very fine material in the center to form an impervious core.

The outlet structures are of concrete on solid rock foundation. In the low dams at Lockington, Taylorville, and Huffman, the spillways are part of the outlet structures, as illustrated in Fig. 126. In the two higher dams at Germantown and Englewood,

TABLE LII.—DATA ON MIAMI VALLEY RETARDING BASINS

Name	Area, acres	Capacity, acre-feet	Drainage area, square miles	Height of dam, feet[1]	Length of dam, feet	Sectional area of outlets, square feet	Discharge in maximum assumed flood, second-feet	Earth, cubic yards	Concrete, cubic yards	Cost (1916)
Lockington	4,020	70,000	355	62	6,400	158	9,400[3]	1,135,000	38,000	865,218
Taylorsville	11,000	186,000	778[2]	59	2,980	1,116	55,000	1,235,000	47,000	1,508,674
Englewood	7,930	312,000	651	108	4,660	214	12,000	4,000,000	38,000	1,959,632
Huffman	9,180	167,000	671	58	3,340	705	35,000	1,655,000	40,000	1,299,125
Germantown	3,520	106,000	270	92	1,210	182	10,200[3]	865,000	20,000	640,235

[1] From bed of river to crest of spillway. [2] Excluding area above Lockington Dam. [3] Approximate.

the spillways are separate from the outlet structures. Figure 127 shows the general layout at the Germantown dam. The outlet structures for the three lower dams are comparatively short and do not extend through the entire bases of the dams; whereas for the higher dams the conduits do extend through the bases of the dams.

The two principal features of the design were the dimensioning of the outlets and the reduction of the extremely high discharge velocities at the outlet to safe velocities below.

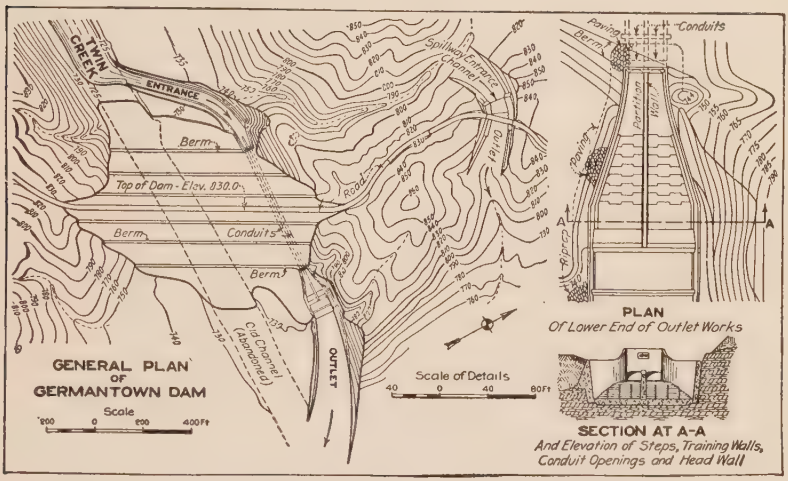


FIG. 127.—General layout of Germantown dam.

Dimensions of Outlets.—The maximum discharge permissible from the outlets of each dam was fixed by the capacities of the channels below the dams, the inflow from uncontrolled tributaries below being accounted for. It was found that a limited amount of channel improvement was more economical than the construction of larger reservoirs, and about one-third of the total cost of the flood protection works was devoted to this purpose.

Various types of outlets were considered and the orifice type was found to have two important advantages over other types:

1. For a given maximum flood and retarding basin capacity, the orifice type of outlet secures the smallest peak outflow rate. 2. For floods less than the maximum, the orifice type of outlet minimizes the extent and time of land flooding in the retarding basin area.

The several steps incident to the design of the retarding basins and outlets were as follows:

1. The first step was to determine the duration of the maximum flood period and the amount of the maximum run-off resulting therefrom. A very comprehensive study of the great storms which had occurred in the Eastern United States lead to the adoption of a 72-hour period of maximum run-off. The maximum 72-hour run-off during the storm of March, 1913, was 7.27 inches over the watershed above Dayton. It was concluded that a safe basis of design would be a 10-inch run-off in 72 hours for the smaller watersheds and 9½ inches for the larger ones. These values of run-off provide for a storm 40 per cent greater than that of March, 1913. The assumed distribution of these run-offs is shown diagramatically in Fig. 128. It will

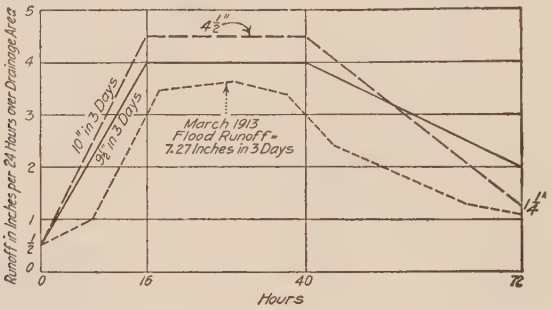


Fig. 128.—Maximum flood run-off curves adopted for the official plan flood.

be observed that the run-off curve starts with an initial rate of 0.5 inch in 24 hours and increases to a maximum rate of 4.5 inches in 24 hours in a period of 16 hours which is the observed time of collection on this watershed. The maximum rate was assumed to remain constant for 24 hours and then to decrease gradually to a rate of 1.25 inches per 24 hours in a period of 32 hours.

2. The second step was to determine the required reservoir capacity. This was done by subtracting the total outflow from the total inflow up to the time that the rates of inflow and outflow would be equal.

The total inflow during the 72-hour period is represented by the area inclosed by the curve in Fig. 128, and the total inflow up to any time is represented by the area under the curve up to the ordinate corresponding to that time.

The outflow from the basin will, of course, vary with the head on the outlet. It was found, however, that if the varying outflow

were discharged from the outlets uniformly during the time required for the water in the basin to reach its maximum elevation, the average rate of outflow would be approximately five-sixths of the maximum rate of outflow.

The relation between inflow, outflow, and storage capacity is shown in Fig. 129. The curve, *ABCDEF*, is the standard inflow curve, similar to that shown in Fig. 128. Curve *AIE* represents the varying rate of outflow under varying heads. At the point of intersection, *E*, the rates of inflow and outflow are equal and the abscissa at *E* represents the time of filling the basin to its maximum elevation for this assumed flood. The areas under the inflow and outflow curves represent the total volumes

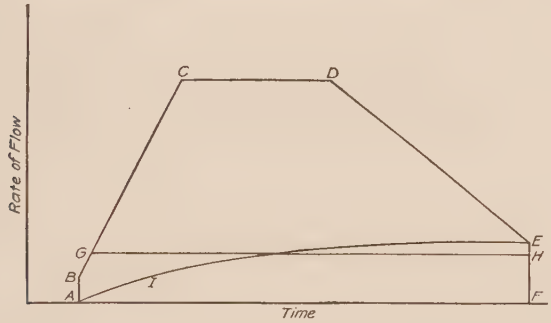


FIG. 129.—Relation between inflow, outflow, and retarding basin capacity when spillway does not overflow.

of inflow and outflow, respectively, and the area *ABCDEI* represents the required capacity of the reservoir. The line *GH* is the curve of equalized flow, the area *ABGHF* being equal to the area *AIEF*.

3. The third step was to determine the height of dam required and the maximum head of water on the outlet. The maximum flood-water elevation obtained in the preceding section fixes both of these quantities.

The maximum head on the outlet is the vertical distance from the maximum water surface in the reservoir to the surface of the water at the discharge end of the outlet. At the end of the outlet, however, the water surface is not at the top of the conduit, because of the drop-off curve. At the Englewood dam where the conduit is 8 feet deep and the maximum discharge velocity about 50 feet per second, the water at the end of the outlet would be only 7.68 feet deep.

4. The fourth and last step was the determination of the size of outlet required to pass the required discharge. The area of the outlet cross-section was determined from the formula,

$$A = \frac{Q}{c\sqrt{2gh}}, \text{ in which } c = \sqrt{\frac{1}{1 + \frac{2gL}{C^2R}}} \qquad (39)$$

In these formulas, C is the Chezy coefficient for conduits, c is the coefficient of discharge for conduits, H is the effective head in feet, L is the length of the conduit in feet, R is the hydraulic radius of conduit section in feet, and the other symbols have the usual significance.

The dimensions of the two types of outlet structures used are shown in Figs. 130 and 131.

The above procedure was applied to each of the five reservoirs, and the final values obtained for each factor are summarized in Table LIII.

TABLE LIII.[1]—DATA USED IN DESIGNING THE OUTLETS FOR THE MIAMI VALLEY RETARDING BASINS

	Retarding basins				
	Germantown	Englewood	Lockington	Taylorsville	Huffman
1. Area of watershed, square miles.	270	651	255	878	671
2. Total flood run-off, inches on watershed	10	10	10	10	10
3. Duration of flood run-off, hours.	72	72	72	72	72
4. Total inflow, acre-feet	144,000	347,200	121,200[2]	474,000[3]	340,000
5. Maximum permissible outflow rate, acre-feet per hour	826	990	744	4,550	2,890
6. Maximum permissible outflow rate, cubic feet per second	10,000	12,000	9,000	55,000	35,000
7. Approximate total outflow, acre-feet	50,160	59,400	44,640	273,000	173,400
8. Maximum storage, acre-feet	93,840	287,800	76,560	201,000	166,600
9. Maximum head, feet	80	91.5	55	40	41
10. Ratio: $\frac{\text{Maximum inflow rate}}{\text{Maximum outflow rate}}$	3.3	6.6	3.4	1.9	2.1

[1] Abstracted from Table 9, *Technical Report* 7, Miami Conservancy District.

[2] Allows for storage in Loramie State Reservoir.

[3] Includes outflow from Lockington Basin and allows for storage in Lewistown State Reservoir.

The minimum size of outlet conduit was governed largely by the length of time required for the basins to empty after a flood period, since it is important that the basins be ready to receive

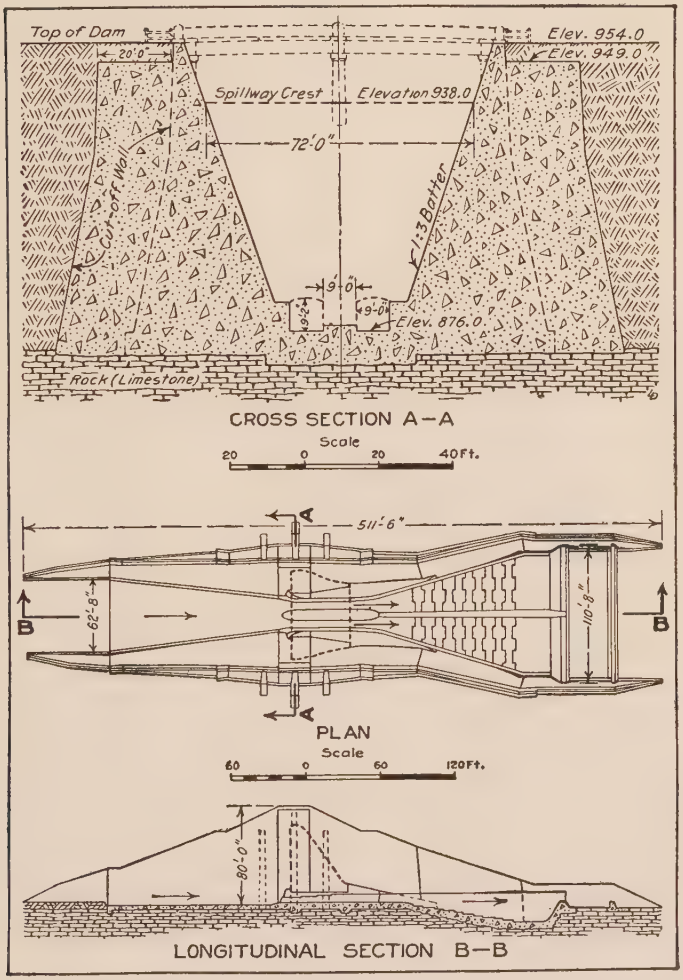


FIG. 130.—Combined outlet and spillway structure at Lockington dam.

the waters from a second flood closely following the first one. To this end, the 28 storms between 1893 and 1917 were studied, assuming the dams to have been in existence. Inflow, outflow, area-depth, and outflow-depth curves were constructed for

each basin. The data obtained from the curves for the Germantown basin are shown in Table LIV.

The Hydraulic Jump.—When discharging under maximum head, the velocity of discharge from the conduits was computed to be from 50 to 60 feet per second. To avoid excessive erosion in the channels below the dams and possibly erosion of the dams themselves, it was necessary to dissipate the energy of the swiftly flowing water as it left the conduits, and thereby reduce the velocity to a safe value. Various methods of accomplishing this reduction in velocity are in use. The energy of the water

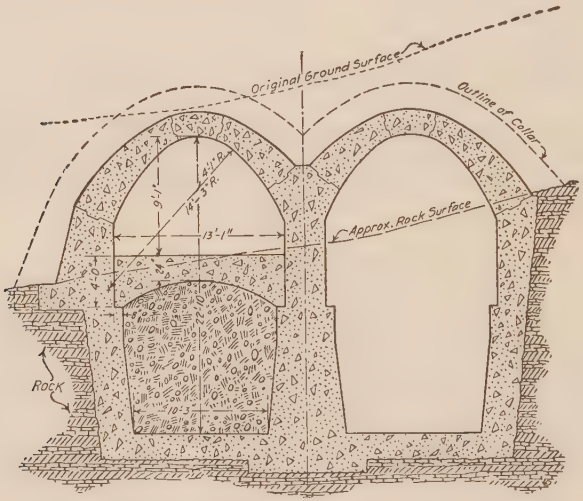


Fig. 131.—Section through Germantown dam outlet conduits.

flowing over the spillways of the Gatun and Bassano dams, Panama, is dissipated by piers placed at the foot of the ogee spillways. A grating of vertical I-beams is used for this purpose below the Waldeck Dam in Germany. A large spraying device is used at the by-pass channel from the Catskill Aqueduct into Croton Lake to reduce to a safe velocity the discharge of 1,500 cubic feet per second at a velocity of 70 feet per second.

After an exhaustive study of these and other methods, and an extensive series of laboratory experiments on models constructed to a one-sixteenth scale, the engineers adopted the "hydraulic jump" method as most nearly meeting the requirements of their problem. The hydraulic jump is the name given to the abrupt rise which occurs in the water surface when a shallow stream

TABLE LIV.—FLOODING CONDITIONS IN GERMANTOWN RETARDING BASIN DURING THE PERIOD 1893 TO 1917 ASSUMING DAM TO HAVE BEEN IN EXISTENCE

The Various Floods Are Arranged in Order of Size

Date of flood crest	Maximum elevation of water surface	Maximum storage, acre-feet	Maximum outflow, second-feet	Area flooded, acres	Days required to empty
Mar. 25, 1913.....	805.2	73,400	9,340	2,950	7.4
Mar. 23, 1898.....	772.6	15,900	6,820	940	2.9
May 2, 1893.....	766.8	10,720	6,260	750	2.3
Oct. 7, 1910.....	765.3	9,640	6,120	730	2.3
Feb. 28, 1910.....	764.9	9,380	6,080	710	2.3
Mar. 6, 1897.....	763.5	8,330	5,920	680	2.0
Feb. 1, 1916.....	763.0	8,040	5,880	670	1.6
July 9, 1915.....	762.8	7,940	5,850	660	2.6
Mar. 14, 1907.....	762.7	7,860	5,850	660	2.0
Feb. 27, 1912.....	761.5	7,060	5,710	630	1.7
Feb. 15, 1908.....	761.4	6,995	5,700	630	2.1
Mar. 30, 1912.....	759.7	5,940	5,500	580	2.0
Mar. 27, 1904.....	759.3	5,690	5,450	560	1.6
Mar. 2, 1908.....	759.1	5,575	5,440	560	1.5
Feb. 25, 1909.....	758.1	4,975	5,300	530	1.7
Mar. 6, 1908.....	757.5	4,680	5,250	510	1.7
Jan. 5, 1907.....	756.8	4,310	5,150	490	1.5
Jan. 18, 1910.....	756.6	4,240	5,140	480	1.7
Mar. 27, 1906.....	756.3	4,080	5,090	470	1.6
Jan. 20, 1907.....	755.7	3,830	5,020	450	1.5
Jan. 22, 1904.....	755.1	3,555	4,950	440	1.6
Jan. 2, 1916.....	755.1	3,555	4,950	440	1.2
Jan. 13, 1916.....	754.9	3,510	4,950	430	1.0
Mar. 1, 1903.....	754.6	3,370	4,880	425	1.3
Mar. 19, 1908.....	753.0	2,795	4,670	380	1.1
Jan. 8, 1913.....	750.7	2,110	4,370	290	1.2
Mar. 27, 1916.....	742.5	620	3,100	110	.9
May 7, 1916.....	735.5	92	2,050	40	.6

moving at a high velocity strikes a deeper, slower moving stream. The standing wave caused by the impact largely dissipates the energy through internal friction. The type of outlet structure which would produce the most satisfactory hydraulic jump and the one adopted for the Miami dams is shown in Fig. 132. There are four special characteristics of this design:

1. The sides of the channel beyond the ends of the conduits are gradually expanded and so shaped that there will be continuous contact between the spreading water and the sides, and that the depth and velocity will be uniform across the channel. These conditions are necessary for a stable and uniform jump.

2. The floor through this portion of the outlet structure is inclined and constructed in steps. This slope is limited only by the parabolic path of the water issuing as a jet from the conduit. The water flowing down this slope cuts under the water in the pool at the foot of the slope. The steps in the bottom reduce the velocity of the water materially before it enters the jump.

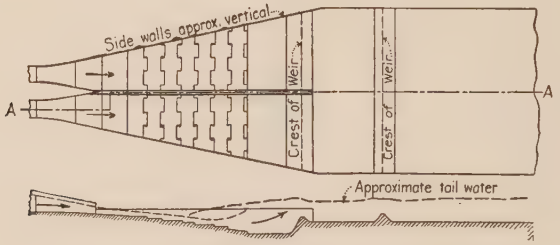


FIG. 132.—Adopted type of outlet structure.

3. A submerged weir is constructed far enough below the foot of the inclined portion to permit a hydraulic jump to form for varying discharges. The position of the jump varies with the discharge and the depth of flow above the jump. The crest of the weir is a little lower than the low-water surface, and its height is not less than one-half the depth of the water above the weir at maximum discharge. The function of this weir is to maintain a sufficient depth of water above it to produce the jump. It is this depth of water above the weir, rather than the weir itself, which causes the jump. If the required depth is not maintained, the jump will not occur.

4. A second weir is constructed a short distance below the first. Without this second weir, the jump is lost at low stages of the tail water, and the water is deflected into the air in the form of a sheet. A second weir as high or higher than the first tends to maintain the height of the tail water and thus to produce the desired jump.

The dimensioning of the outlet structure at the Lockington dam is shown in Fig. 130, and a view of the hydraulic jump in action during a moderate flood is shown in Fig. 133.

FIG. 133.—The hydraulic jump at low and high heads.

The theory of the hydraulic jump is based on Newton's second law of motion, which is, that the rate of loss of momentum must be equal to the unbalanced force acting on the moving mass to retard its motion. This law expressed as a formula[1] for rectangular channels is:

$$D_2 = -\frac{D_1}{2} + \sqrt{\frac{D_1^2}{4} + \frac{2D_1V_1^2}{g}} \qquad (40)$$

in which,

D_1 = depth above the jump.
D_2 = depth below the jump.
V_1 = velocity above the jump.
g = acceleration of gravity.

This formula can be changed to the following form:

$$J^2 + J = 4\frac{H_1}{D_1} \qquad (41)$$

where $J = \frac{D_2}{D_1}$

and $H_1 = \frac{V_1^2}{2g}$ = velocity head above the jump.

The relation between J and $\frac{H_1}{D_1}$ is shown diagrammatically by the curve in Fig. 134. The formula and curve show that the

[1] This formula derived by Prof. S. M. Woodward in *Technical Report* 7 of the Miami Conservancy District.

greater the ratio of the velocity head to the depth above the jump, the greater will be the jump; and consequently the greater the dissipation of the energy in the water in passing through the jump. As a result of this dissipation of energy, the velocity of about 50 feet per second at the conduit outlet is reduced to about 4 feet per second in the stilling pool below the jump.

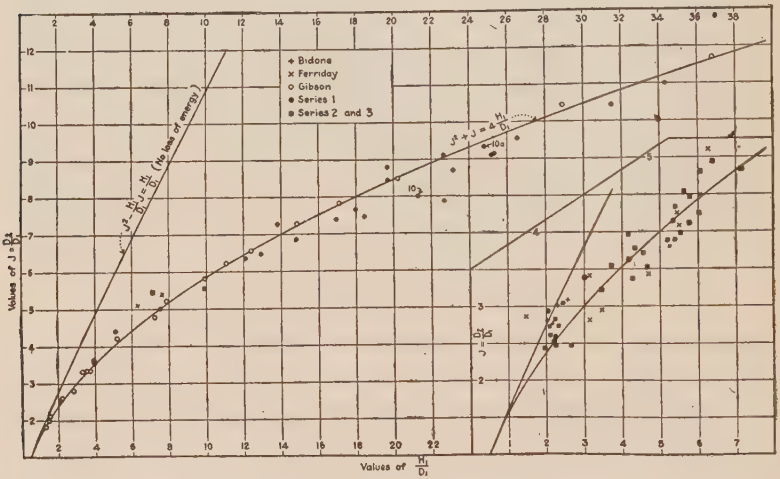


FIG. 134.—Diagram showing agreement between momentum formula and experimental data.

Scioto Valley Flood Relief.[1]—The storm of March, 1913, also caused much damage in the Scioto Valley, Ohio, and resulted in the formation of the Franklin County Conservancy District. The plan of this district provided for the construction of two reservoirs and for channel improvement; and, although the plan was not carried out because of an adverse court decision, a brief statement of its principal features will be of value.

Since flood protection was to be obtained partly by reservoirs and partly by channel improvement, one valuable study was for the determination of the economic balance between reservoir construction and channel improvement. The problem was to provide for a flood run-off of 200,000 second-feet at a minimum cost. Plans were made for enlarging the channel and constructing levees to take care of flood flows of 35,000, 40,000, 50,000,

[1] The information given in this section is compiled from the "Report to the Franklin County Conservancy District on Flood Relief for the Scioto Valley," by ALVORD and BURDICK.

60,000, 100,000, 150,000, and 200,000 second-feet, respectively. The cost under each of these plans is shown diagrammatically by the curve marked "Channel Improvement" in Fig. 135. Plans were then made for the construction of reservoirs of different capacities and the cost of each was estimated and plotted to form the curve marked "Reservoirs" in the figure. The curve marked "Total Cost" represents the sum of the cost of each reservoir project and the cost of the channel improvement that would be required along with it. From the last curve it is seen

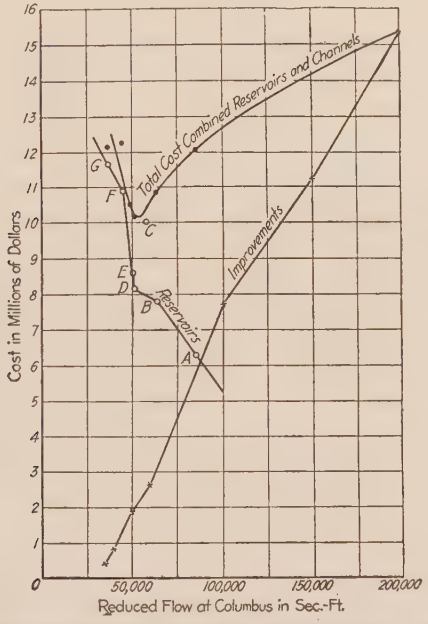


FIG. 135.—Cost balance between projects for reservoirs and channels to protect against 200,000 second-feet.

that project *D*, costing $10,125,000, is the cheapest. Of this sum, $8,210,000 is for reservoir construction and $1,915,000 for channel improvement. Under this plan the channel at Columbus would carry 50,000 cubic feet per second, and the two reservoirs would retain a flow of 150,000 cubic feet per second. This plan provides for a flood run-off 45 per cent greater than that of March, 1913; and the margin of safety allowed in the reservoirs and in the freeboard of the levees permits a flood run-off of 250,000 cubic feet per second before the levees would be overtopped.

The dams at the two reservoirs were to be constructed of earth. The larger reservoir, which is on the Scioto River near Dublin, Ohio, would be 143 feet high and 980 feet wide at the base, and would hold 255,925 acre-feet of water at spillway level. The dimensions of the smaller dam, which is on the Olentangy River near Delaware, were height, 82 feet, and thickness of base, 850 feet, and the capacity of the reservoir at spillway level was 163,202 acre-feet.

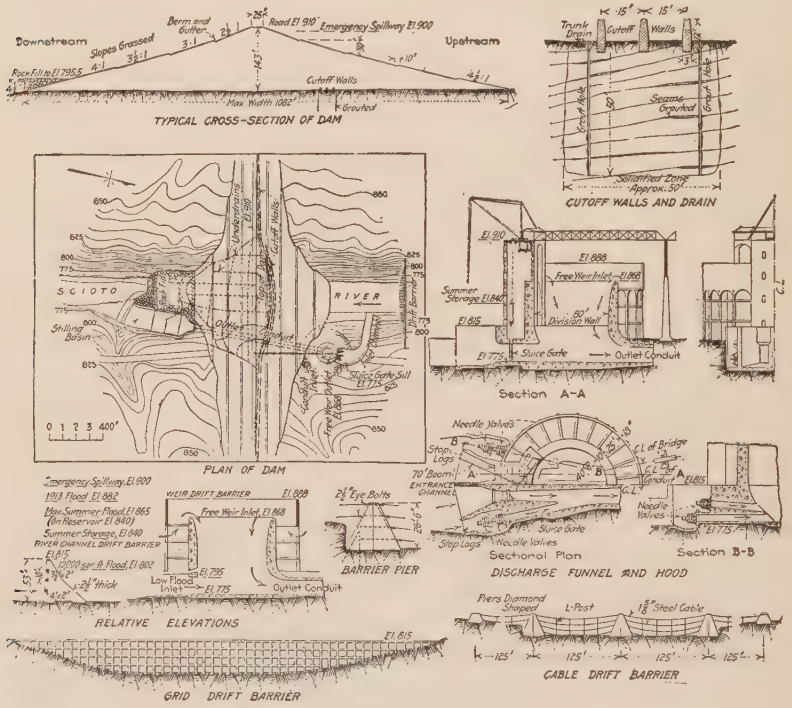


FIG. 136.—Design of Dublin dam.

The Dublin dam was planned with a threefold purpose in mind; primarily for flood protection, secondarily for additional water supply for the city of Columbus, and thirdly for the control of the small, summer floods which frequently cause much damage to the farming lands in the valley. The last two purposes preclude the use of detention dams such as were used on the Miami project. The design of the outlet structure is shown in Fig. 136. In the base of the outlet structure two sluice gates and four needle-valve gates are provided for the regulation of small,

summer floods and for ponding the water required for the city of Columbus during the dry summer months. The elevation of the water surface is regulated by the needle-valve gates. In the fall, all the gates would be opened and the reservoir emptied in readiness for the winter and spring floods. These gates open into two conduits proportioned so as to limit the rate of discharge below the dam.

About 90 feet above the top of the gates, there was provided a free weir inlet in the form of a circle, 80 feet in inside diameter as shown in Fig. 136. This circular weir discharged into the two conduits under the dam. Surrounding the weir, there was a large reinforced concrete hood, 150 feet in diameter, supported on columns. The bottom of this cylindrical hood was a little below the level of the weir to keep out drift. Its top extended 20 feet above the crest of the weir. The minor details of the design are shown in Fig. 136.

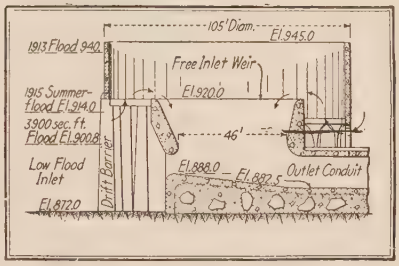


FIG. 137.—Delaware Basin outlet.

The Delaware dam was not required to store water for Columbus, and, therefore, no gates were provided in the small openings in the base of the dam. The outlet structure for this reservoir is shown in Fig. 137.

Flood Control in Los Angeles County, California.—The rainfall and run-off conditions along the Pacific coast are different from those in other parts of the United States, due to climate, topography, and geographic location; and, therefore, the problem of flood control is somewhat different. On the Pacific coast most of the annual precipitation occurs during the months from December to March. The moist winds from the Pacific Ocean, sweeping easterly over the coastal plain, drop only a small amount of rain over the plain; but when they come in contact with the cold mountain slopes, which rise to an elevation of 8,000 to 9,000 feet, and are forced upward, the moisture is precipitated in torrents. Thus, at Los Angeles the average annual precipitation for a 38-year period is only 15.81 inches,[1] whereas in the mountain areas near-by it amounts to from 50 to 60 inches.

[1] *Eng. News-Record*, p. 273, Feb. 10, 1916.

During the storm of Feb. 18 to 21, 1914, the rainfall at San Pedro on the coast was 2.03 inches; at Los Angeles, 20 miles inland, it was 7.07 inches; and at Mount Lowe, 11 miles from Los Angeles, it was 19.20 inches. This excessive precipitation falling on mountain slopes of from 200 to 1,400 feet per mile, runs off in torrents, carrying with it a large amount of boulders, gravel, and sand. Where the streams debouche upon the plains, this material is deposited, and in the course of time immense

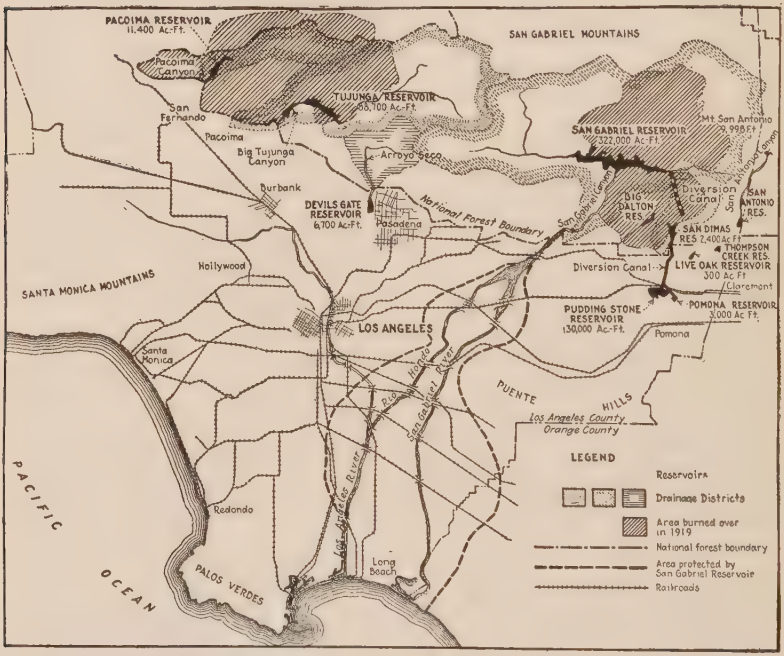


FIG. 138.—Los Angeles County Flood-control District, showing location of reservoirs.

débris cones with slopes of about 50 feet to the mile have been built up. Flowing over these cones, the rivers spread out, forming several channels which change their position frequently and flow over the plains, doing a large amount of damage.

The Los Angeles County Flood-control District was organized to control the floods from the Los Angeles, San Gabriel, and Hondo rivers. The flood prevention plans now being carried out consist of a number of reservoirs in the mountain canyons. The locations of these reservoirs are shown in Fig. 138, and data concerning them are given in Tables LV and LVI. The dams

listed in the former table have been constructed, and the others will be constructed as rapidly as possible. The large reservoir on the San Gabriel River is planned for a threefold purpose, namely, flood control, irrigation, and water supply. This reservoir is to be operated as follows: During the rainy winter months a continuous discharge will be permitted at such a rate that the entire discharge will be absorbed by the débris cones,

TABLE LV.—DAMS BUILT BY LOS ANGELES COUNTY FLOOD PROTECTION DISTRICT, 1918 TO 1924

Dam	Location	Reservoir capacity, acre-feet	Over-all height, feet	Drainage area, square miles	Cost of project	Date completed
Devil's Gate....	Arroyo Seco at Pasadena	6,700	130	30	$406,000	1920
San Dimas.....	Tributary to San Gabriel Ry.	2,400	130	17	501,000	1922
Live Oak.....	North of La Verne	500	70	3	168,000	1922

and the ground water supply thus replenished. It is estimated that 500,000 acre-feet of water can be thus absorbed. The gates will then be closed and the reservoir allowed to fill. The stored water will be used for irrigation and water supply during the summer months. In the early winter, the reservoir will be emptied in readiness for the winter floods.

In addition to the reservoirs mentioned, numerous small check dams or barriers have been constructed in the channels of the mountain streams to retard the velocity of the water.

Flood Control on the Sacramento and San Joaquin Rivers, California.—The method of controlling the floods on these rivers, as established by the Sacramento and San Joaquin Drainage District, consists of:[1]

1. Confining to the river all the water that can be carried safely between well-located levees of reasonable height.

2. Allowing the water in excess of the river capacity, thus created, to go over the bank at selected points under control.

3. Preventing the general inundation of lowlands by confining this surplus water between embankments that will conduct it to a reentry into the lower reaches of the river or into Suisun Bay.

[1] GRUNSKY, C. E., "Some Factors Affecting the Problem of Flood Control," *Trans. Amer. Soc. Civ. Eng.*, p. 1488, 1922.

TABLE LVI.—LOS ANGELES COUNTY FLOOD-CONTROL PROJECTS APPROVED MAY 6, 1924

Projects	Capacity, acre-feet	Cost	Type of dam	Height, feet
1 Retarding dams, Pomona, San Antonio Canyon and spreading grounds	.....	$ 450,000		
2 Thompson Creek reservoir	520	146,000	Two earth dams	72 and 30
3 Big Dalton reservoir	1,200	555,000	Multiple arch	155
4 San Gabriel reservoir (200 square miles drainage)	200,000	25,000,000	Concrete, gravity arched in plan	500
5 Sawpit Canyon reservoir	350	241,000	Concrete, arched 200-ft. radius	140
6 Big Santa Anita reservoir	1,530	586,000	Concrete, constant angle arch	235
7 Sierra Madre canyon (small concrete and wirebound check dams)	.....	150,000		
8 Eaton Canyon reservoir	1,033	603,000	Concrete, constant angle arch	220
9 Altadena storm drain	.....	250,000		
10 Pickens canyon (one small arch and masonry or wirebound check dams)	.....	100,000		
11 Verdugo wash, Sycamore canyon, Glendale and Los Angeles city (open concrete conduits)	.....	450,000		
12 Wirebound dams in Dunsmuir and other canyons	.....	150,000		
13 Haines canyon (wirebound rock check dams)	.....	100,000		
14 Big Tujunga reservoir	14,000	1,600,000		
15 Pacoima reservoir and resort damage (30 square miles drainage)	11,600	1,720,000	Concrete, constant angle arch	375
16 Work on lower Ballona creek	.....	250,000		
17 Work at Virginia City at Long Beach	.....	500,000		
18 Work upon the lower east San Gabriel River near Artesia	.....	600,000		
19 Puddingstone reservoir and channel to San Dimas Wash	20,000	1,750,000	1 concrete 3 earth	170 20, 53, and 63
20 Incidental expenses	.....	99,000		
Total	.....	$35,300,000		

The lower portion of the Sacramento River, below the mouth of Stony Creek, has banks built up by the deposition of sediment, which are higher than the lands away from the river, thus forming basins similar to those along the Lower Mississippi. On the west of the Sacramento are the Upper and Lower Colusa Basins and the Yolo Basin; and on the east, are Butte, Sutter, and Sacramento Basins.

On the other hand, the land in the San Joaquin Valley slopes from both sides to the river, and there are no basins similar to those along the Sacramento River.

The construction of storage reservoirs on the Sacramento is not feasible and hence the adoption of the "by-passes" on the lower river. Above Colusa the excess discharge will be allowed to flow over a concrete weir into a by-pass channel through Butte and Sutter Basins. At a point about 26 miles below Colusa, a second weir is provided which discharges into the by-pass through the Sutter Basin. At the mouth of Feather River, the Fremont weir on the right bank permits an overflow into a by-pass channel about 2 miles in width through the Yolo Basin. A fourth weir at Sacramento enables the excess flood waters at this point to flow into the Yolo by-pass. The Sutter by-pass reenters the Sacramento River at the mouth of Feather River near the Fremont weir, so that the Yolo by-pass discharges the entire excess flow into the Lower Sacramento above the mouth of the San Joaquin River. The Butte by-pass is planned to carry safely between levees a discharge of several hundred thousand cubic feet per second. The Tisdale weir below Colusa will add about 35,000 cubic feet per second to the discharge through the Sutter by-pass.

The by-passes, to all intentions and purposes, function as storage reservoirs to handle the excess flood waters, even though the storage is only temporary and the discharge through the lower river below the by-passes is greater than it would be under natural conditions of flow.

The flood prevention works on the San Joaquin River will consist principally of levees, aided to some extent by five reservoirs, constructed primarily for irrigation. The location and capacity of these reservoirs are given in Table LVII. The Don Pedro reservoir has been constructed. Since these reservoirs are for irrigation purposes, their effect will be small in the case

of large flood flows; but they will aid materially in preventing small floods.

TABLE LVII.—RESERVOIR PROJECTS ON THE SAN JOAQUIN WATERSHED

Project	Capacity, acre-feet
Pine Flat Reservoir on Kings River	600,000
Madera Irrigation District Reservoir on San Joaquin River	600,000
Excelsior Reservoir on Merced River	300,000
Don Pedro Reservoir on Tuolumne River	250,000
Melones Reservoir on Stanislaus River	70,000

Proposed Reservoirs on Minnesota River.—Following the destructive flood on the Minnesota River in the spring of 1919, steps were taken toward the prevention of future floods. The plan recommended by the engineer employed by the State Department of Drainage and Waters is so different from any plan which has been adopted for flood control that it is worthy of a brief description in this discussion.

The plan provides for the construction of three reservoirs in the valley of the river itself, since there are no suitable sites for reservoirs on any of the tributary streams. The sites chosen are at Ortonville, Montevideo, and New Ulm (Fig. 139). The tops of the proposed dams are only a few feet above present flood heights, as indicated in the profile shown in Fig. 140. The combined storage capacity of the three reservoirs would be 18,500,000 cubic feet. The area which would be flooded by the three reservoirs is only one-third of the area which would be protected by them. The proposed project would not give protection against the maximum storm which would occur on an average of once in 15 years. The damage caused by floods in this valley is simply the flooding of agricultural lands, and the cost of a greater degree of protection is not warranted. The area subject to flooding is comparatively small since the flooded portion is only about 1 mile in width, and according to the engineer's report, this narrow width of valley precludes the economic use of channel improvement and levees.

The proposed method of operating the reservoirs is to fill the upper reservoirs first and empty them last. No water is to be

discharged from any reservoir until it is full or until the stage in the reservoir next below is falling.

FIG. 139.—Minnesota River watershed.

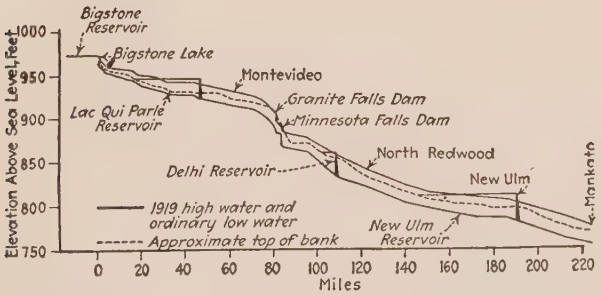


FIG. 140.—Profile of Minnesota River and proposed flood-control reservoirs.

Colorado River Flood Control.—The most important flood control project under consideration at the present time (1925) is that of the Lower Colorado River. There is urgent need for a

large regulating reservoir on this river, first for the prevention of floods, and second for irrigation purposes. Since the maximum capacity of a reservoir here is much greater than that demanded for the above-mentioned purposes, power development will add a third function to be served by such a reservoir.

The principal areas menaced by the floods from the Colorado are the Imperial and Coachella Valleys in California. These valleys are in the Salton Basin, a depression over 2,000 square miles in area which is below the level of the sea. About two-thirds of the area in this basin is rich agricultural lands under cultivation. In 1905, the Colorado River broke through into the Salton Basin, and for over 2 years poured its entire volume into this area, filling it to a depth of about 80 feet. The cost of returning the river to its old channel was about $5,000,000. A large, storage reservoir above is the only means of preventing an eventual recurrence of this disaster. The annual discharge at the end of the canyon section of the river is estimated at 17,500,000 acre-feet. Storage of between 5,000,000[1] and 6,000,000 acre-feet would confine the flow of the Colorado to less than 25 per cent of its present maximum flood discharge.

During a period of low water flow in 1915, the entire discharge of the river was diverted and used for irrigation in the Imperial Valley. Since that time 400,000 more acres have been placed under irrigation, and there are still about 150,000 acres in the valley to be brought under cultivation. To insure water to these lands during dry years and to enable their expansion, a storage reservoir is essential. It is estimated that about 11,000,000 acre-feet of storage will be required for this purpose.

Since the control of the Colorado is an international as well as an interstate problem, the project will have to be undertaken by the United States Government. The United States Reclamation Service has made a thorough study of the problem and recommended to Congress the construction with government funds of a reservoir at or near Boulder Canyon, the government to be reimbursed by the revenues from leasing the power privileges incident thereto.

Table LVIII gives the capacities of reservoirs of different heights at the proposed site. If 5,000,000 acre-feet of storage be reserved for flood control and 5,000,000 acre-feet for silt storage,

[1] DAVIS, A. P., "Flood Problems in the Arid Regions," *Trans. Amer. Soc. Civ. Eng.*, p. 1485, 1922.

there will still remain 21,600,000 acre-feet for irrigating the 2,000,000 acres of agricultural lands in the lower Colorado basin, and this flow would develop about 700,000 horsepower, which would pay for the construction of the dam.

TABLE LVIII.[1]—CAPACITY OF BOULDER-CANYON RESERVOIR

Computed from Original Plane-table Sheets; Scale, 2 Inches Equal 1 Mile

Contour elevations	Area, acres	Capacity, acre-feet	Contour elevations	Area, acres	Capacity, acre-feet
700	.....	.....	1,100	67,740	10,153,000
750	2,350	58,750	1,150	84,110	13,949,250
800	7,950	316,250	1,200	105,100	18,679,500
850	15,260	846,500	1,250	127,660	24,498,500
900	21,620	1,818,500	1,260	131,000	26,000,000
950	29,160	3,088,000	1,280	142,000	28,600,000
1,000	39,690	4,609,250	1,300	152,000	31,600,000
1,050	53,160	7,130,500			

Note.—The canyon walls extend up to above the 2,000-foot contour, or about 700 feet higher than the last one for which capacity is calculated.

[1] *Eng. News-Record*, p. 743, May 4, 1922.

Conclusions.—The best plan for controlling the flood of a stream is that one which will give the desired degree of protection for the least expenditure; hence, in any specific instance, the best plan can be determined only after all the practical methods have been investigated and the cost of each of them estimated. Although the methods of flood control given in this chapter and the two preceding ones are discussed separately, they are not to be considered as separate solutions of the flood problem, since most flood-control projects, constructed or proposed, consist of an economic combination of two or more of them. Where a combination is evidently the correct solution, the extent to which each should be used can be determined by striking an economic balance as explained in the discussion of the Scioto River project.

There are two general classifications of all flood-control measures. In the first class are those projects which hasten the flow of the water from the watershed. This follows from channel improvement and levee construction. In the second class are those projects which retard the flow of the flood waters by storage or detention basins. Assuming that these two general

methods are equally feasible, both physically and economically, the latter is the more desirable for the following reasons:

1. All visible evidence of flood danger is removed, since the volume of flow and stage of stream are not such as to cause alarm or anxiety to the public—a condition which does not exist when large volumes of flood waters are carried between levees at a stage many feet above the level of the adjacent lands, even though the levees are amply strong and high.

2. The flow of the stream is more uniform even than under natural conditions, thus permitting a more satisfactory use of the stream flow for water supply, sanitation, and other purposes.

3. The flood-control works are concentrated at a few points, where the structures can be more easily and safely maintained than where such structures extend throughout the valley.

CHAPTER XIV

EXCAVATING MACHINERY

The engineer cannot intelligently plan work involving excavation and make estimates of the cost of construction unless he has a knowledge of the machines used for excavating and of their performance in different soils and under different working conditions.

It is impossible in the short space allotted to treat the subject of excavating machinery in a comprehensive manner. Only those machines which are suitable for the construction of ditches and levees are considered, and their descriptions are limited to those features of construction which are essential to an understanding of their operation. The kind of material which each type of machine is best adapted to excavate, and the conditions under which each is most efficient are considered of special importance, since they have a direct bearing on the cost of construction.

Excavating machinery may be classified in a number of ways, depending upon the basis of distinction. In this chapter only two broad classifications are used: (1) the *dry-land excavator* which operates from the surface of the ground and can be used only where the ground is solid enough to support the machine; and (2) the *floating excavator* which operates from the water and can be used in excavating both solid and soft material.

DRY-LAND EXCAVATORS

Dry-land excavators may be considered under two general classes, based on the method of operation: (1) machines hauled by horses; and (2) machines operated by steam, gas, oil, or electricity. Machines of the first class are light and cheap and can be placed on the job at small expense, but the cost of excavating with them is several times that of power excavators; hence, they are applicable especially to work involving a small yardage. On the other hand, machines of the second class are heavy, high

in first cost, and expensive to haul to the construction site and to assemble. Where the volume to be excavated is large, however, these disadvantages are more than offset by the lower cost of excavating.

Drag Scraper.—The drag scraper is the simplest form of scraper and consists of a steel pan or scoop with a cutting edge, a bail for attaching a team of horses, and handles at the rear which are used in guiding the scraper while loading and emptying it.

The field of usefulness of this scraper, as well as that of all scrapers hauled by teams, is limited to excavation in dry materials, since it cannot be used under water or in material so soft as to mire the horses. In the past, the drag scraper has been used extensively in the construction of levees and shallow drainage ditches, where the material has been borrowed from pits near the levees or wasted on the banks of the ditches. In recent years, the fresno scraper, described later, has largely supplanted the drag scraper because it is a more efficient implement, since it uses more horsepower and less man power.

The drag scraper can be used economically only for short hauls, and is most efficient for haul distances up to 150 feet. The haul distance is the distance from the place where the material is loaded to where it is dumped and does not include the distance traveled by the team in turning. The economical limit of haul for the largest size is considered by most contractors to be about 200 feet. The amount of material which a team and drag scraper can move in a 10-hour day will vary from 35 to 70 cubic yards for hauls from 25 to 200 feet.

Drag scrapers are manufactured in several sizes, varying in capacity from 3½ to 5½ cubic feet, and cost about $10. The rated capacities must be decreased at least 25 per cent in estimating their performance, since earth occupies more space after it is loosened, and since a full load is rarely hauled.

Fresno Scraper.—The scoop of the fresno scraper is long and narrow, and to its ends are attached adjustable runners which extend forward of the cutting edge. The material is dumped by pushing the handle forward, thus throwing the weight on the runners, until the bottom of the scoop is vertical, thereby raising the cutting edge and allowing the material to spill out. In this position the scraper is hauled back to the borrow pit. An illustration of this scraper is shown in Fig. 141.

This scraper is manufactured in three sizes. The No. 1 has a 5-foot cutting edge and a hauling capacity of about 15 cubic feet; the No. 2, a 4-foot cutting edge and a capacity of about 12 cubic feet; and the No. 3, a 3½-foot cutting edge and a capacity of about 10 cubic feet. The struck-measure capacity of each of these sizes is less than the hauling capacity, since in any soil but dry sand the material will be heaped up above the top of the scoop and will extend some distance ahead of the cutting edge and will be pushed along, making the effective capacity of the scoop greater than the measured capacity. The cost of the fresno scraper is about $25 at factory.

The 4-foot fresno, which is the size ordinarily used, requires three mules and one driver. The driver loads and dumps the scraper, and the only additional help needed in a fresno outfit

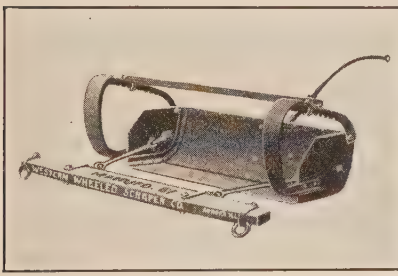


FIG. 141.—Fresno scraper.

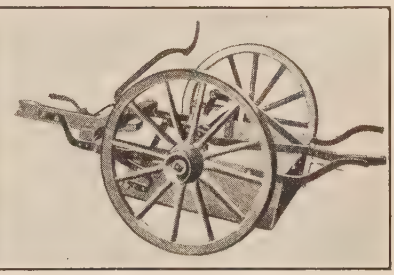


FIG. 142.—Two-wheel scraper.

is four mules and one driver for the plow. The output varies directly as the haul distance. To operate a fresno outfit economically, the number of fresnos should be sufficient to keep the plow team busy, or else the plow teams should be used partly for hauling.

The economic limit of haul is generally placed at about 300 feet, although this is a variable quantity, since it depends upon the size of the outfit used, the price of labor, and the efficiency of the outfit. For the average fresno outfit, there is a distance between 300 and 350 feet beyond which the fresno is not as economical an implement as the wheel scraper. The fresno scraper is especially useful in moving surface material.

Two-wheel Scraper.—Figure 142 shows a view of a two-wheel scraper. It is simply a large drag scraper carried on a pair of wheels. The advantages of the wheel scraper over the drag

and fresno scrapers are: (1) the somewhat larger loads which can be carried by it; (2) the longer hauls which can be made with it because the load is carried upon wheels instead of being dragged over the ground; and (3) the larger number of wheel scrapers which can be used with a given number of mules, since only two mules are required to haul a wheel scraper, whereas three are required for a fresno, and sometimes four.

The disadvantages of the wheel scraper as compared to the drag and fresno scrapers are: (1) that a snatch team and driver are required for loading, and two additional men, one for loading and the other for dumping; and (2) that for a given number of mules in an outfit, more drivers are required for wheel scrapers than for fresnos, as well as more scrapers.

Two-wheel scrapers vary in rated capacity from 7 to 16 cubic feet, and actually carry about 4½ to 11 cubic feet of earth "in place." The capacity and cost of the several sizes in which the wheel scraper is manufactured are given in Table LIX.

TABLE LIX.—CAPACITY AND COST OF TWO-WHEEL SCRAPERS

No.	Rated capacity, cubic feet	Cost as of Feb. 1, 1925
0	7	
1	9	$59.50
2	11	65.00
2½	14	69.50
3	16	81.50

The economical minimum haul distance for two-wheel scrapers depends upon the size of the outfit, the scale of wages, and the efficiency of the field organization. It is the same as the maximum haul distance of the drag or fresno scraper. It is that distance at which the number of wheel scrapers employed is just sufficient to keep the snatch team busy, and at which the cost of output is the same as that of fresnos for the same haul distance. Table LX shows comparative estimates of the cost of working the same outfit on fresnos and on wheel scrapers.

The economical maximum haul distance for the two-wheel scraper is about 500 feet. At about this distance it becomes more

economical to use either the four-wheel scraper or the elevating grader and wagons.

TABLE LX.[1]—COMPARATIVE ESTIMATES OF THE COST OF WORKING THE SAME OUTFIT ON FRESNOS AND ON WHEELERS

	With teamsters at 20 cents an hour		With teamsters at 40 cents an hour	
	Six fresnos	Eight wheelers	Six fresnos	Eight wheelers
Outfit:				
4 mules on plow, 18 mules on fresnos, 2 mules extra; 24 mules, total	$15.00	$15.00	$15.00	$15.00
Add one-fifth for Sundays and lost time	3.00	3.00	3.00	3.00
1 foreman	4.00	4.00	6.00	6.00
1 plow holder	2.50	2.50	4.50	4.50
1 four-line driver	2.50	2.50	4.50	4.50
6 fresno drivers	12.00	.....	24.00	
8 wheeler drivers	.....	16.00	.....	32.00
1 snatch-team driver	.....	2.00	.....	4.00
1 loader	.....	2.50	.....	4.50
1 dump man	.....	2.00	.....	4.00
1 extra laborer	2.00	2.00	4.00	4.00
Total cost	$41.00	$51.50	61.00	81.50
Operating cost ratio	100 to 126		100 to 133	
Less 1 driver if 1 wheeler is not used	.....	2.00	.....	4.00
Total cost	41.00	49.50	61.00	77.50
Operating cost ratio	100 to 121		100 to 127	
Less 2 drivers if 2 wheelers are not used	.....	4.00	.....	8.00
Total cost	41.00	47.50	61.00	73.50
Operating cost ratio	100 to 116		100 to 120	

[1] HARRISON, J. L., Highway Engineers, Bureau of Public Roads, "The Economical Use of Wheel Scrapers," *Public Roads*, vol. 5, no. 10.

Four-wheel Scraper.—The four-wheel scraper, a view of which is shown in Fig. 143, consists of a steel frame mounted on two trucks, a large pan or bucket suspended by chains, and mechanism for lowering and raising the pan and for dumping the excavated material. The chains at the rear end of the pan are fastened to the frame, whereas those at the front end are wound on spools which are operated by sprockets and chains from the rear wheels. A lever releases the spool clutches and allows the front end of the empty pan to drop to the loading position. When the pan is filled, a foot lever engages the clutches on the axle with the clutches on the rear wheels, and the forward movement of the wheels revolves the spools and winds up the chains. When the front end of the pan has been raised to the required

height, the clutches are thrown out automatically and the pan is held in this position while being hauled to the dump. A dumping lever releases the pan lock which engages the pan and end gate, and allows the pan to swing freely forward, suspended from the spools by the two pan lifting chains. At the same time, the end gate tilts backward allowing the material to pass out.

This scraper is manufactured in the ¾ and 1¼ cubic yard sizes and costs $350 and $625, respectively, at factory. It is hauled either by horses, using a snatch team or tractor for loading, or by some form of tractor. The scraper is most efficiently used in trains of from four to six, drawn by one tractor. It is adapted to hauls over 300 feet, and has been used very success-

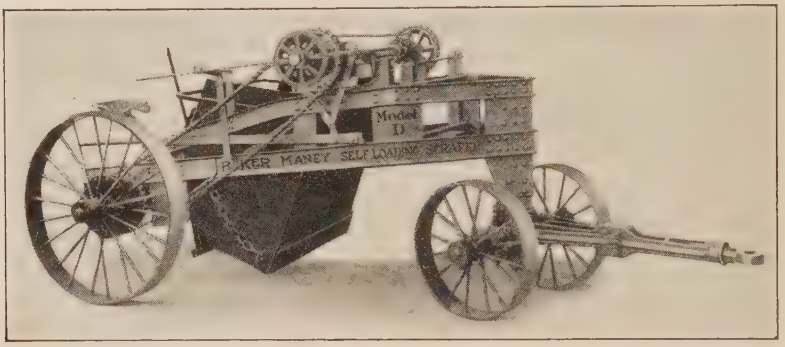


FIG. 143.—Four-wheel scraper.

fully in the construction of levees and dams. Using the larger size of scraper in trains of four behind a tractor, it should be possible in a 10-hour day to haul 630 cubic yards a distance of 300 feet, or 315 yards a distance of 1,320 feet. Tractors of from 20 to 40 horsepower are required to load and haul these scrapers, depending on the number of scrapers in the train. One man can load and dump three scrapers. When there are four to six scrapers in a train, two men are required to operate the scrapers in addition to the tractor operator.

Elevating Grader.—The essential features of this machine are a plow and a belt-conveyor, fastened to a strong frame which is mounted on two trucks, as illustrated in Fig. 144. The grader plows a furrow from 12 to 14 inches wide and 10 to 12 inches deep. The plow turns the furrow directly onto the belt-conveyor which carries the dirt upward and outward from the machine and dumps it into wagons moving parallel to the grader. The

conveyor lengths vary from 15 to 30 feet and their widths from 36 to 40 inches. The 15-foot conveyor is ordinarily used for dumping into wagons. The longer conveyors are used where the dirt is cast directly into place without the use of wagons.

The grader is pulled either by horses or by tractor. From 16 to 20 horses are generally used. The number of wagons needed will depend upon the length of haul; but there should always be enough of them to keep the grader in continuous operation. For a haul of 300 feet, 5 wagons are recommended, and 1 additional wagon for each 100 feet of additional haul. Thus, for a haul of 1,000 feet, 12 wagons should be used with 1

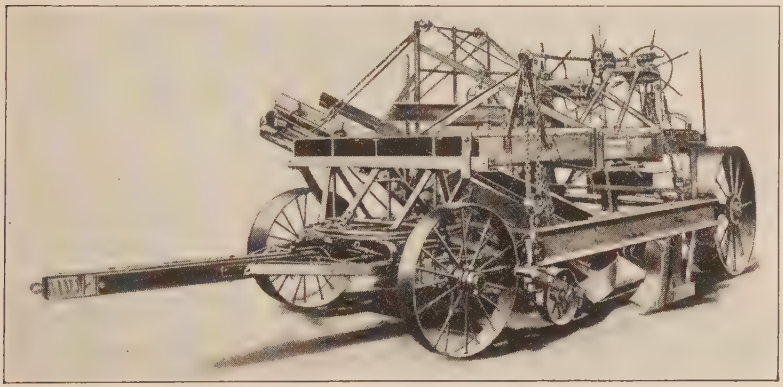


FIG. 144.—Elevating grader.

grader. Generally only one operator is required in addition to the drivers.

This machine is capable of loading from 50 to 80 wagons an hour, and the manufacturers guarantee that it will excavate 100 cubic yards an hour when plowing in ordinary earth. The price of a standard elevating grader is $1,900, as of Feb. 1, 1925; and the cost of dump wagons is about $230.

This machine has been used extensively in the construction of dams and levees; and for hauls over 300 feet is more economical than the wheel scrapers or fresno scraper, provided the yardage to be moved is sufficiently large to warrant the use of the more expensive equipment.

Drag-line Scraper Excavator.—Although this excavator has come into prominence only within the last few years, it is probably the most generally used of all dry-land excavators at the

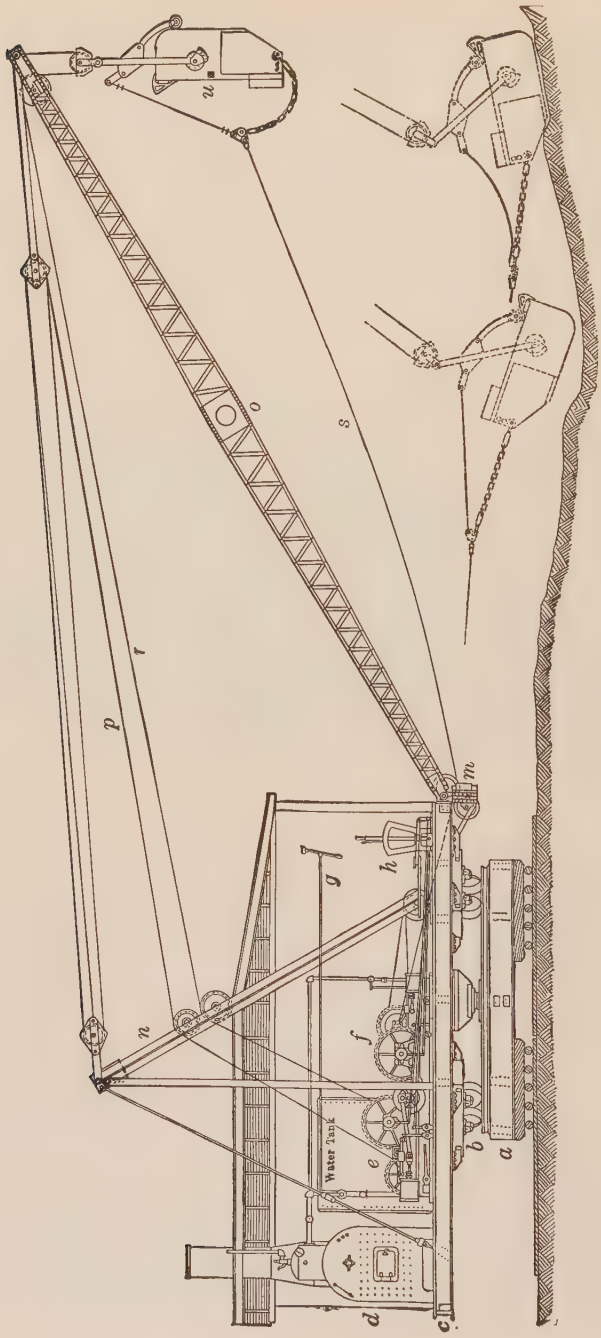


FIG. 145.—Detail view of drag-line excavator.

(A.B. McDaniel "*Excavating Machinery.*")

present time. Large ditches and levees can be constructed with it at less cost than with any other type of dry-land excavator. It is also applicable to the construction of earth dams. It is manufactured in both the stationary and the revolving or rotating types, but the latter is the one commonly used.

The essential parts of the revolving type of drag-line scraper excavator are: (1) the bucket; (2) the boom from the upper end of which the bucket is suspended; (3) the A-frame which supports the upper end of the boom; (4) the engine and the cable drums for operating the bucket; (5) the engine used in swinging the bucket by revolving the upper platform to which the lower end of the boom is hinged; (6) the power equipment for operating the engines; (7) the upper platform upon which the boom, A-frame, engines, and power equipment are mounted; (8) the turntable under the upper platform by means of which it is revolved; and (9) the lower platform which is mounted either on trucks, caterpillar traction, skids and rollers, or a special walking device for moving the excavator ahead. All of these are shown in Fig. 145.

Operation.—In the construction of a ditch, the drag-line scraper excavator begins at the lower end and works upstream, the machine moving along the center line of the ditch. In constructing levees, it operates from the berm between the levee and the borrow pit. Four men are required to operate a steam-power excavator—an operator who controls the entire operation by levers and brakes at the front end of the platform, a fireman who fires the boiler and oils the machinery, and two laborers who work on the ground and see to the proper placing of the track sections which are moved forward as the work progresses. The two laborers are not needed when caterpillar traction or the walking device is used. The bucket is loaded by hauling on the drag line, at the same time manipulating the hoisting wire to give the bucket the proper digging angle. When the bucket is filled, the clutch is released and the drum held by the brake. The hoisting drum is then clutched and the bucket is raised—the drag line being payed out by easing up on the brake—until it is at the height necessary for dumping, when it is swung over the dump by the swinging engine. The bucket is dumped by releasing the tension on the drag line.

Cost.—The costs of rack-and-pinion-swing, rotating, drag-line excavators are given in Table LXI which is reproduced from

Bulletin 300 of the Bureau of Public Roads, United States Department of Agriculture.

TABLE LXI.—COSTS OF RACK-AND-PINION-SWING, ROTATING, DRAG-LINE EXCAVATORS, AS OF 1921

Size of bucket, cubic yards	Length of boom, feet	Kind of power	Style of mounting	Approximate cost
½	30	Oil engine	Caterpillar	$11,000
¾	40	Steam	Caterpillar	14,800
¾	40	Oil engine	Caterpillar	18,650
1	40	Oil engine	Walker	15,700
1½	45	Steam	Skid and rollers	20,850
1½	45	Steam	Caterpillar	29,200
2	60	Steam	Walker	31,000
2	60	Steam	Skid and rollers	26,700
2	60	Steam	Caterpillar	38,300
2½	85	Steam	Skid and rollers	38,300
3	60	Steam	Walker	36,500
3½	100	Steam	Skid and rollers	46,900
3½	100	Steam	Trucks	56,800
3½	125	Steam	Trucks	71,000
5	155	Steam	Trucks	97,500

TABLE LXII.[1]—OUTPUT OF DRAG-LINE EXCAVATORS

Size of bucket, cubic yards	Length of boom, feet	Economical size of job,[2] cubic yards	Output per month, with double shift, cubic yards
1	40	250,000	18,000
1½	50	300,000	25,000
2	60	400,000	30,000
2½	65	500,000	35,000
3	70	800,000	40,000
3½	80	1,000,000	50,000
4½	100	1,500,000	70,000

[1] Reproduced from *Bull.* 300, Bureau of Public Roads, United States Department of Agriculture.

[2] By economical size of job is meant the yardage below which the machine cannot be installed and operated without appreciably increasing the cost per cubic yard of excavation.

The output of this machine depends upon the depth of cut, the angle of swing, the efficiency of the operator, the character

of the material to be excavated, the cross-section of the ditch or levee, and the time lost because of adverse weather conditions and breakdowns. Table LXII gives values for the output of the several sizes, and the economic relation between the size of the bucket and the total volume to be excavated. These values are the average output, including lost time. Under favorable conditions they may be exceeded from 50 to 100 per cent.

Non-rotating Drag-line Excavator.—This type of excavator differs from the revolving type mainly in that there is only one platform, which is stationary, and that the boom is pivoted at its lower end. It is manufactured in the smaller sizes only and is applicable principally to the construction of small drainage ditches.

One make which is used rather extensively is designed to straddle the ditch. The operating platform is on a large, steel frame, mounted on four large wheels, each 5 feet in diameter and 3 feet wide. The machine is moved forward by means of a cable attached to a "deadman." The boom is 40 feet long and the bucket capacity is $\frac{5}{8}$ cubic yard. It is operated by a 45-horsepower gasoline engine, weighs about 12 tons, and costs about $5,500. The output of this machine is about 300 cubic yards per shift.

The distinctive feature of another type is a boom made in two sections—one above the other—which are hinged at their lower ends. The top of the upper section is guyed to the A-frame; the lower section is raised and lowered by the hoisting line which passes over sheaves in the upper ends of the sections and is fastened to the rear end of the bucket, which is pivoted through an arm to the under side of the lower section. The drag line is fastened to the front end of the bucket. The machine is operated by first releasing the tension in the hoisting line, thereby lowering the lower section of the boom and the bucket until the latter is on the ground in the ditch. The bucket is then loaded by pulling it toward the machine with the drag line. With the drag-line drum held by brake, the hoisting line is then used to raise the lower section of the boom and the bucket until the latter is high enough to dump. The boom as a unit is then swung about its bull-wheel mounting until the bucket is over the dump. By releasing the tension in the drag line, the bucket is dumped.

Grapple Excavator.—This type of dry-land machine differs from the drag-line excavator only in the kind of bucket used.

The scraper bucket is replaced by either a clam-shell or an orange-peel bucket. The former is composed of two hinged jaws or shells, whereas the latter has three or four segments which form a hemispherical bowl when closed. The operation of these buckets will be more fully described later under the subject of floating grapple dredges on which they are mainly used. The bucket is supported by two cables—the opening line and the closing line—which pass over sheaves in the top of the boom and thence to the drums.

This machine is made in both the revolving and stationary types, and is applicable to the construction of ditches and levees where the material is soft enough to permit the bucket to penetrate it sufficiently to secure a full load.

Dry-land Dipper Dredge.—The principle of operation and the general features of construction of this machine are similar to

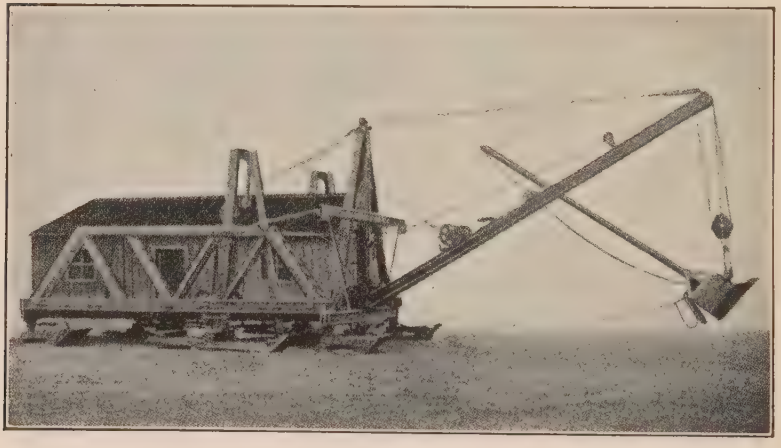


Fig. 146.—Walking dredge.

those of the floating dipper dredge which is described later. It differs from the floating dredge mainly in its mounting. Whereas the machinery of the floating dredge is mounted on the deck of a boat, that of the dry-land dredge is mounted on a platform which spans the ditch. The platform, in turn, is mounted either on trucks which move on a track or is equipped with a walking device. The dry-land dredge is lighter than the floating dredge and is more easily assembled and dismantled, which is its chief advantage.

This dredge is manufactured in the ½- and 1-yard sizes and is operated by an internal combustion engine. The ½-yard machine has a span of from 14 to 18 feet, and a boom from 24 to 30 feet in length, and digs 12 feet below the surface. The 1-yard machine has a span of from 23 to 60 feet and a boom length of from 36 to 50 feet, and digs 14 feet below the surface. The output of the 1-yard dredge under average conditions is about 500 cubic yards per 10-hour shift. A dry-land walking dredge is illustrated in Fig. 146.

Another type of dipper dredge has a double boom similar to that of one of the drag-line machines previously described. The bucket is attached to an arm which is pivoted at its mid-point to the end of the lower section of the boom. The bucket is loaded by rotating the arm about this pivot. The bucket has a capacity of 1¾ yards, and the boom is from 30 to 50 feet in length.

Tower Excavator.—Another type of excavator which has been used effectively in the construction of levees is the tower excavator. This machine consists essentially of two towers, a cableway between the towers, a drag bucket, and the operating machinery.

The head tower is about 85 feet high and is located just back of the levee being constructed, as illustrated in Fig. 147. The tail tower is about 45 feet high and is located on the opposite side of the borrow pit. The distance between the two towers is generally about 600 feet. Both towers are mounted on trucks which rest upon specially designed track, as illustrated in Fig. 148, and are moved along the length of the levee as the work progresses. The towers of the smaller excavators of this type are mounted on caterpillars.

There are two general arrangements of the cables between the towers. In one arrangement there is a taut cable between the towers upon which travels a carriage, which is moved back and forth by an endless line operated by one of the drums of the main engine. A line attached to the bail of the bucket passes over a sheave on the carriage and over a sheave at the top of the head tower, and thence to a second drum on the main engine. By means of this line, the empty bucket is lowered to the ground and, after it is loaded, raised and held in the air while being moved to the levee. The bucket is loaded by another cable, leading from the front end of the bucket to a third drum on the main engine. Another line fastened to the back of the bucket and running to

the conveyor drum enables the bucket to be dumped by means of a haul-down rope.

In the second arrangement, the cable upon which the carriage travels is slack and the bucket is attached to the carriage by two rods which are hinged to the bucket as shown in Fig. 147. The bucket is raised and lowered by decreasing or increasing the slack in the cable. By proper manipulation of this cable, the

FIG. 147.—Head tower of tower excavator.

bucket is loaded by means of a drag cable fastened to the front end of the bucket and passing over a sheave on the lower part of the head tower to a drum on the main engine. The loaded bucket is dragged over the ground surface of the borrow pit, and when near the levee is raised by the carriage cable and is dumped by releasing the tension in the drag line. Another cable is fastened to the back of the bucket and passes over a sheave at the top of the tail tower and back to the head tower. By means of this cable the empty bucket is hauled back to the point of digging.

The buckets used on tower excavators vary in capacity from 3 to 8 cubic yards, although the 6-yard bucket is probably used more than any other size. The first cost of tower excavators is from $50,000 to $75,000.

The crew of this type of excavator comprises two foremen and riggers, two head-tower operators, one tail-tower operator, one fireman, one oiler, 10 trackmen and dressers, and one team and teamster.

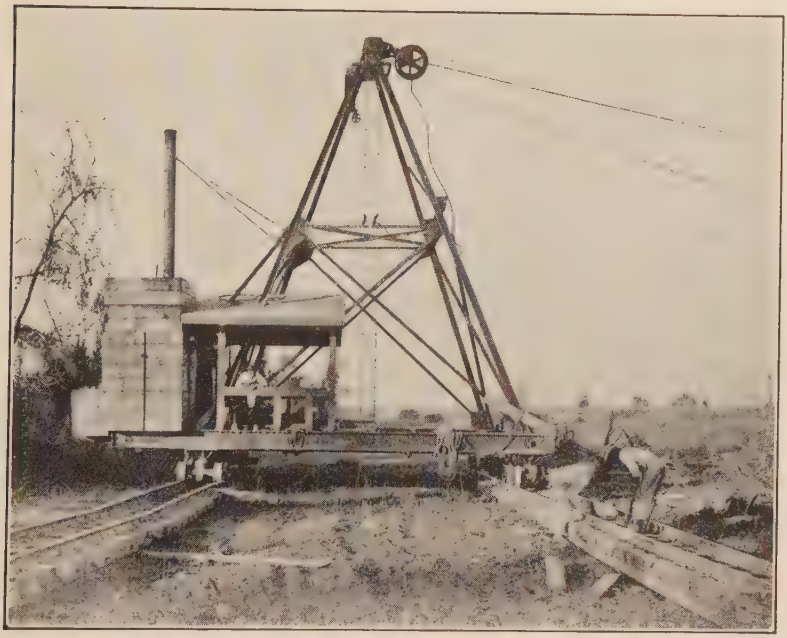


FIG. 148.—Tail tower of tower excavation.

The average output of a machine of this type is from 2,000 to 3,000 cubic yards in a 10-hour shift. Under favorable working conditions, these values may be exceeded 50 to 100 per cent for short periods.

It is generally agreed that the tower excavator will build levees at a field cost of from 8 to 12 cents a cubic yard, exclusive of clearing and grubbing.

Templet Excavator.—There are two kinds of excavators which make use of the templet feature. One is adapted to

the excavation of ditches and the other to the construction of levees.

The *ditching machine* has a vertical templet or frame of steel, whose lower end has the shape of the cross-section of the ditch. The templet can be raised and lowered through the rear end of the platform. A bucket with a cutting edge at each end and fitted with a plunger head is pulled back and forth along the under side of the templet. The machinery raises and lowers the templet, operates the bucket, and moves the excavator forward as the work progresses. Both steam and internal combustion engines are used, although the latter is more common.

The machine is manufactured in two sizes: one with a templet which will excavate a ditch with a bottom width of 3 feet and a depth up to 8 feet; and the other, a bottom width of 6 feet and a depth up to 12 feet. It is operated by lowering the templet through the platform to the surface of the ground. The bucket is moved along the templet and cuts a thin slice of earth, which it carries up the opposite side of the templet frame until it is over the spoil bank, when trips on the frame push the plunger head through the bucket, thus forcing out the material and moving the plunger head to the other end of the bucket. The bucket is then lowered, loaded, carried up the other side of the templet, and dumped. As the material is excavated, the templet is lowered, until the desired depth of ditch is obtained. The machine is then moved forward a distance equal to the width of the bucket, and the process repeated.

The principal advantage of the templet excavator is that it cuts a smooth channel with correct cross-section.

The *levee-building machine* operates from the berm between the borrow pit and the levee. The templet in this case is in front of the machine and is suspended by cables from a boom. The bucket is pulled along the under side of the templet, under the platform, and up a frame at the back which extends over the levee. The bucket does not make a cut on the return trip.

Both of these machines are efficient, but their use is limited to excavations in soil free from stumps, boulders, and rock.

Wheel Excavator.—The wheel excavator has been developed for constructing small ditches where a dipper or drag-line machine cannot be used. Its digging equipment consists of from 8 to 12 broad, scoop-shaped buckets attached to the circumference of a large wheel which revolves either on a central axle or on

anti-friction wheels, which in turn are attached to a steel frame pivoted to the rear end of the platform and extending beyond it. The latter frame is lowered, together with the wheel and buckets, as the work proceeds, until the required depth of cut has been obtained. As the wheel revolves, each bucket cuts a slice of earth. As the loaded buckets pass over the top of the wheel, the material slides out onto a belt-conveyor which carries it to the spoil bank. The operating machinery is on the front end of the platform. The front end of the platform is supported on a truck with broad-tired wheels, whereas the rear end is mounted on caterpillars. Ordinarily, internal-combustion engines are

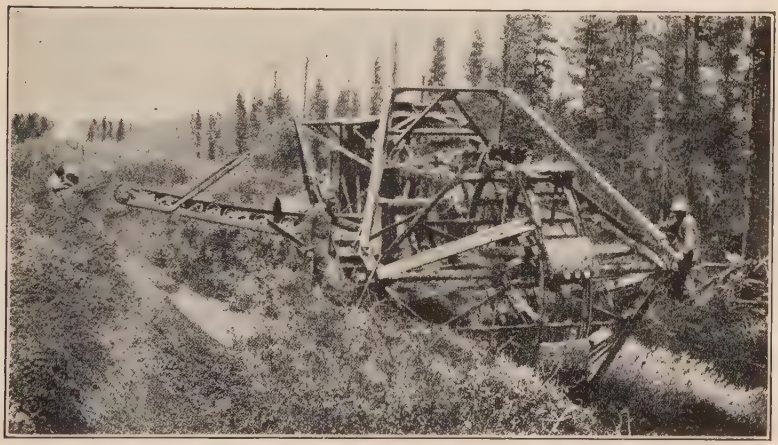


FIG. 149.—Wheel excavator.

used for power. The cost of this machine ranges from $7,500 to $20,000, according to size. The smallest size weighs 18 tons and will excavate a ditch 3 feet deep and 3½ feet wide at the top; the largest size weighs 50 tons and will excavate a ditch 5½ feet deep and 12 feet wide at the top. The cutting speed varies from 2 to 10 feet per minute, depending upon the working conditions and the efficiency of operation.

Another type of wheel excavator has a narrow wheel and small buckets attached to two chain belts. Cutting blades, inclined from the circumference of the wheel to the ends of the axle, form the sides of the ditch. The material cut by these blades falls to the bottom of the ditch and is removed by the buckets. This machine is illustrated in Fig. 149.

FLOATING EXCAVATORS

Floating Dipper Dredge.—The floating dipper dredge has been used more extensively than any other type of machine in the construction of drainage ditches. The large amount of drainage work which has been done in this country, and especially in the Middle West, is due to the early development of this machine. It is nothing more than the familiar steam shovel mounted on a boat instead of on wheels. Numerous types of this machine are manufactured, but the essential principles are the same in all of them.

The principal parts of the floating dipper dredge are the dipper or bucket, the dipper handle, the boom, the A-frame, the main engine and boiler, the swinging engine, the spuds, and the hull.

Dipper.—The dipper is an open bucket with a hinged flap bottom, a cutting edge, and a bail to which the hoisting line is fastened. The bucket is drawn back toward the hull to the loading position by means of a backing line. It is loaded by pulling on the hoisting line which forces the bucket forward into the face of the material to be excavated.

The capacity of dippers varies from ½ to 15 cubic yards. For ditch construction and river work, the 5-yard size is usually the maximum; however, two 10-yard dippers were used in the construction of the Cape Cod Canal, and two 15-yard dippers were used on the Panama Canal.

Dipper Handle.—The lower end of the handle is rigidly attached to the back of the dipper. The upper end rests upon, and moves over, a pinion in the boom near its midpoint. The pinion meshes with a rack on the under side of the handle. Two friction-band wheels are keyed to the pinion shaft by means of which the handle is held fast at any point in its length, but is free to rotate about that point. The handle is usually about two-thirds the length of the boom.

Boom.—The boom is a relatively large member of wood or steel, designed to withstand the severe bending and torsional stresses to which it is subjected. The lower end is attached to a large horizontal wheel by means of which the boom and dipper are swung. The upper end of the boom is guyed to a yoke at the top of the A-frame by adjustable cables. The hoisting line passes over a sheave in the top of the boom and thence to the bail of the dipper. The length of boom is from one and a

half to two times the width of the hull, depending upon the type of spuds used, and varies from 20 feet for the ½-yard dipper to 95 feet for the 5-yard dipper.

A-frame.—The frame is shaped like the letter "A" and is usually composed of structural-steel members. The lower end of the frame is bolted to the hull in front of the hoisting drum. When mounted vertically, the frame is guyed by rods to the rear corners of the hull. When inclined backward, back legs extend from the top of the frame to the hull. The end of the boom is guyed to the ends of a yoke which revolves on a large pin in the head of the A-frame.

Main Engine.—The hoisting and backing lines are operated by the main engine which is of the horizontal, double-cylinder, non-reversible, friction-drum type. On the smaller dredges, where the deck space is limited, the main engine is also used for swinging the boom. This is effected by a long shaft, with drums at its ends, geared to the engine. The drums may be keyed to the shaft and the swinging cables wound on them in opposite directions so that one winds as the other unwinds, or the drums may be free on the shaft and operated by friction clutches. The boilers generally used are either of the locomotive or of the Scotch marine type.

Swinging Engine.—The double, reversible type of engine is used for the swinging of the boom. It is compound-geared either to a single drum or to a long shaft with two drums as explained in the last section. The swinging cable passes around a large turntable, mounted on a heavy base at the front end of the hull, to which the lower end of the boom is fastened. For large dredges, a stationary type of turntable is used, and the boom is swung by a horizontal beam fastened to it.

Spuds and Spud Engines.—Spuds are devices for holding the hull still while the dredge is working. There are three of them, one at the center of the stern and one on each side near the front end. The rear spud is always vertical and consists of a heavy timber with a pointed iron shoe at its lower end which moves in a box or guide frame attached to the hull. When lowered, the pointed end is forced into the bottom of the ditch or river and prevents the rear end of the hull from moving. The spud is raised by a cable operated by the spud engine. The side spuds may be either vertical or inclined. Vertical side spuds are similar to the rear spud, except that they have a foot or platform at

their lower ends to give a larger bearing area and to prevent the spuds from sinking into the mud when the boom is swung from side to side. The dredge shown in Fig. 150 is equipped with

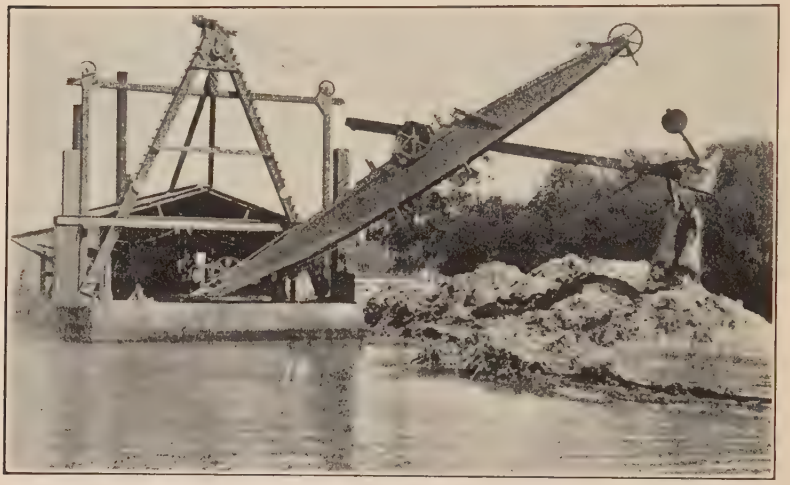


FIG. 150.—Dipper dredge with vertical spuds.

FIG. 151.—Dipper dredge with telescopic bank-spuds.

vertical spuds. Inclined spuds are fastened to the A-frame and their lower ends rest on the banks, hence they are called bank spuds. They are usually telescopic to conform to the changes in the height of the banks. This type of spud is limited to the

construction of ditches which are slightly wider than the hull of the dredge. The dredge shown in Fig. 151 is equipped with telescopic bank spuds.

On the smaller dredges, the spud drums are fitted with friction clutches and are operated by the main engine in connection with the hoisting and backing drums. On the larger dredges, there is usually a separate engine for operating each spud.

Hull.—The hull or boat is a heavily constructed box of wood or steel, thoroughly stiffened with bulkheads and trusses to withstand the severe stresses to which it is subjected when the dredge is in operation. Its dimensions depend on the length of the boom, size of dipper, and size of operating machinery. The relation of these factors is shown in Table LXIII.

Operation.—The crew of the dipper dredge consists of an engineer, a craneman, a fireman, and from two to four laborers. The engineer operates the hoisting, backing, and swinging engines, while the craneman, who is stationed at the foot of the boom, operates the dipper-handle frictions on the boom and opens the bucket.

The dipper dredge is usually operated downstream in order to have the necessary depth of water to float it. When working upstream, dams are constructed behind the dredge to pond the water.

Cost.—The approximate costs of dipper dredges are given in Table LXIII. These values are exclusive of the hull and the cost of assembling. The cost of operating the dipper dredge will depend upon the size and type of dredge used, the kind of soil to be excavated, the magnitude and location of the work, the price of labor and fuel, delays due to breakdowns, and the efficiency of the operators. With so many factors involved it is difficult to give costs. Table LXIII, however, gives the outputs of the several sizes of dredges under average conditions, and, knowing the labor and fuel costs for any locality, the cost of assembling the dredge, and the overhead charges, the cost per cubic yard can be approximated.

Applicability.—The dipper dredge is especially adapted to the construction of drainage ditches in swampy, timbered country, and has been used more than any other type of excavator for this class of work. It is a hard-digging machine and can excavate any material from silt to loose rock, and can remove boulders and stumps.

TABLE LXIII.[1]—DIMENSIONS, LIMITATIONS OF OPERATION, COSTS, AND OUTPUT OF FLOATING DIPPER DREDGES

Size of dipper, cubic yards	Length		Depth it will dig below water, feet	Height it will dump above water, feet	Distance, center of hull to center of dump, feet	Size of hull		Approximate cost, 1919	Minimum economical yardage, cubic yards	Average output per month with double shift, 52 shifts, cubic yards
	Boom, feet	Dipper handle, feet				Bank spuds, feet	Vertical spuds, feet			
½	20	14	6	5 to 7	18 to 20	48 × 10 × 4				
	22	15	6½	6 to 8	20 to 22	48 × 11 × 4				
	25	17	7½	7 to 9	22½ to 25	48 × 12 × 4				
¾	25	19	8 to 10	6 to 8	20 to 25	54 × 12 × 4½	54 × 18 × 4½			
	30[2]	21 to 22	9 to 12	8 to 10½	25 to 31	56 × 14 × 4½	56 × 20 × 4½	.....	100,000	20,000
	35	23 to 25	10 to 14	9½ to 12	28 to 35	56 × 16 × 4½	56 × 20 × 4½			
1	30	24	11	9 to 12	29 to 34	65 × 14 × 5	65 × 22 × 5			
	35	25 to 27	10 to 13	11½ to 15	33 to 40	66 × 16 × 5	65 × 23 × 5			
	40[2]	28 to 30	14 to 14½	12 to 17	37 to 44	70 × 17 × 5	70 × 25 × 5	$12,600	300,000	27,000
	45	31 to 33	15½ to 16	15 to 18	41 to 49	70 × 19 × 5	70 × 27 × 5			
	50	34	17½	18 to 21	45 to 50	70 × 22 × 5	70 × 30 × 5			
	55	37	19	21 to 24	50 to 55	80 × 24 × 5	80 × 32 × 5			
	60	40	20½	24 to 27	55 to 60	80 × 26 × 5	80 × 34 × 5			
1½	35	26	11	14	35 to 40	72 × 16 × 5				
	40	28 to 30	13 to 14	12 to 16½	35 to 45	75 × 18 × 6½	75 × 26 × 6½			
	45	31 to 33	15 to 16	15 to 19	40 to 49	75 × 20 × 6½	75 × 28 × 6½			
	50[2]	34 to 36	16 to 17	17 to 21½	45 to 51	80 × 22 × 6½	80 × 30 × 6½			
	55	35 to 39	17 to 18½	20 to 24	50 to 60	80 × 24 × 6½	80 × 32 × 6½	18,600	500,000	37,000
	60	38 to 40	18 to 20	22 to 26	55 to 65					
	65	38 to 43	18 to 21½	23 to 29	60 to 70					
	70	46	18	26	70 to 75	.....	96 × 34 × 7			
2	40	28	13	10 to 13	35 to 40	85 × 20 × 6½	85 × 28 × 6½			
	45	31	14½	13 to 15	40 to 45	85 × 22 × 6½	85 × 30 × 6½			
	50	34 to 37	16 to 17½	15 to 22	45 to 55	85 × 24 × 7	85 × 30 × 7			
	55	37 to 40	17½ to 19	18 to 24½	50 to 61	85 × 26 × 7	85 × 32 × 7			
	60[2]	40 to 43	19 to 20½	21 to 27	54 to 66	90 × 28 × 7	90 × 34 × 7	25,200	800,000	47,000
	65	43 to 46	20½ to 22	24 to 29½	59 to 71	90 × 30 × 7	90 × 36 × 7			

	70	46	22	27 to 30	65 to 70	100 × 32 × 7	100 × 40 × 7			
	75	49	23½	30 to 33	70 to 75	100 × 34 × 7	100 × 44 × 7			
	80	52	25	33 to 36	75 to 80	100 × 36 × 7	100 × 46 × 7			
	85	55	26½	36 to 39	80 to 85	100 × 38 × 7	100 × 46 × 7			
	90	58	28	39 to 42	85 to 90	110 × 40 × 7	110 × 48 × 7			
2½	40	28	13	10 to 13	35 to 40	85 × 22 × 7	85 × 30 × 7			
	45	31	14½	13 to 15	40 to 45	85 × 24 × 7	85 × 32 × 7			
	50	34	16	15 to 18	45 to 50	90 × 26 × 7	90 × 34 × 7			
	55	37	17½	18 to 21	50 to 55	90 × 28 × 7	90 × 36 × 7			
	60	39 to 43	18 to 20½	22 to 26½	54 to 66	90 × 29 × 7½	90 × 35 × 7½			
	65	42 to 46	18 to 22	23 to 29	59 to 71	95 × 31 × 7½	95 × 37 × 7½			
	70[2]	46 to 49	20 to 23½	26 to 31½	63 to 76	95 × 33 × 7½	95 × 39 × 7½	31,500	1,000,000	55,000
	75	49 to 52	23½ to 25	30 to 34½	67 to 80	100 × 35 × 7½	100 × 41 × 7½			
	80	50 to 52	22 to 25	30 to 36	75 to 85					
	85	55	26½	36 to 39	80 to 85	100 × 40 × 7½	100 × 48 × 7½			
	90	58	28	39 to 42	85 to 90	110 × 42 × 7½	110 × 50 × 7½			
3	60	40	20	21 to 24	55 to 60	90 × 28 × 8	90 × 36 × 8			
	65	43	21½	24 to 27	60 to 65	90 × 30 × 8	90 × 38 × 8			
	70	46 to 50	23 to 24	27 to 32	63 to 77	100 × 33 × 8	100 × 39 × 8			
	75	49 to 53	24½ to 25½	30 to 34½	67 to 83	100 × 35 × 8	100 × 41 × 8			
	80[2]	52 to 56	26 to 27	33 to 37	71 to 88	105 × 37 × 8	105 × 44 × 8	37,000		
	85	55 to 59	27½ to 28½	34½ to 39½	76 to 94	110 × 40 × 8	110 × 46 × 8			
	90	58	29	39 to 42	85 to 90	110 × 40 × 8	110 × 48 × 8			
3½	60	40	20	21 to 24	55 to 60	90 × 30 × 8	90 × 38 × 8			
	65	43	22	24 to 27	60 to 65	90 × 32 × 8	90 × 40 × 8			
	70	46	23½	27 to 30	65 to 70	100 × 34 × 8	100 × 42 × 8			
	75	49	25	30 to 33	70 to 75	100 × 36 × 8	100 × 44 × 8			
	80	52	26½	33 to 36	75 to 80	100 × 38 × 8	100 × 46 × 8			
	85	55	28	36 to 39	80 to 85	110 × 40 × 8	110 × 48 × 8			
	90	58	29½	39 to 42	85 to 90	110 × 42 × 8	110 × 50 × 8			
4	75	53	25½	29½ to 34	68 to 83	105 × 37 × 8½	105 × 43 × 8½			
	80	56	27	32 to 36½	71 to 88	110 × 39 × 8½	110 × 45 × 8½			
	85[2]	59	28½	34½ to 39	76 to 94	110 × 41 × 8½	110 × 47 × 8½	50,000	2,000,000	75,000
	90	62	30	37 to 41½	81 to 99	115 × 44 × 8½	115 × 49 × 8½			
5	85	59	28½	34 to 38½	76 to 94	115 × 41 × 9	115 × 47 × 9			
	90[2]	62	30	36½ to 41	81 to 99	120 × 45 × 9	120 × 51 × 9	66,000	3,000,000	80,000
	95	65	31½	39 to 43	85 to 104	125 × 49 × 9	125 × 55 × 9			

[1] Compiled from tables in *Bull.* 300, Bureau of Public Roads, United States Department of Agriculture.

[2] Length of boom ordinarily used with corresponding size of bucket.

The channel excavated by the dipper dredge is rougher than that formed by some other types of machines. It is difficult to obtain a true bottom grade and to secure smooth side slopes. To offset this objection, contractors usually dig a deeper and wider section than that called for in the plans.

As far as possible the engineer should design his ditches to conform to the limitations of operation of the various excavators. In Table LXIII such limitations are given for the dipper dredge.

Floating Grapple Dredge.—The substructure of the floating grapple dredge is similar to that of the floating dipper dredge, and the superstructure is similar to that of the dry land grapple excavator, previously described. The hull is not so heavily braced, however, as that of the dipper dredge, since it does not have to withstand a dipper thrust and since the boom is mounted farther from the front of the hull. The boom may be longer than that used on the land machine, and lengths up to 240 feet are in use. The boom is swung by a swinging engine which operates a cable passing around a bull-wheel, or by means of the hoisting and backing lines. The bucket may be either of the clam-shell or of the orange-peel type. The latter is better adapted to work in stumpy ground and where the material is solid enough not to run out of the bucket; whereas the former bucket is more efficient in digging very soft materials. The bucket is suspended by two lines. The opening line is attached to the cross-head or top of the bucket, and when suspended by it the bucket is open. The closing line is attached to a reeved block on the shaft about which the jaws of the bucket rotate, and the bucket is closed by stressing this line. The reeved type of clam-shell bucket is illustrated in Fig. 152.

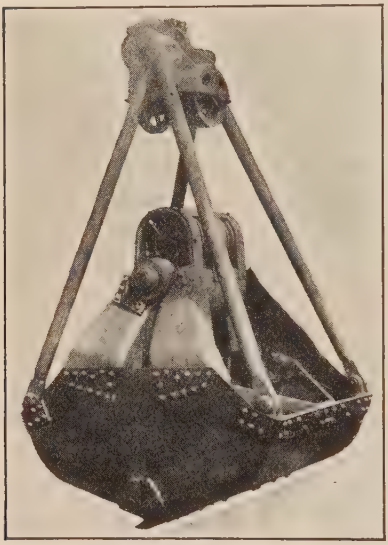


FIG. 152.—Reeved type of clam-shell bucket.

On account of its long boom, the grapple dredge is especially adapted to levee work. It has not been used very much for the construction of ditches, since it is impossible to construct a smooth channel with it. It is at its best in soft mud, mud and sand, and muck. It cannot be used as efficiently in excavating stiff clay. This machine, with a clam-shell bucket, has been used extensively for levee building in California.

Hydraulic Dredge.—The hydraulic or suction dredge has been used in the construction of levees and in the improvement of river channels. It is the only type of power machine

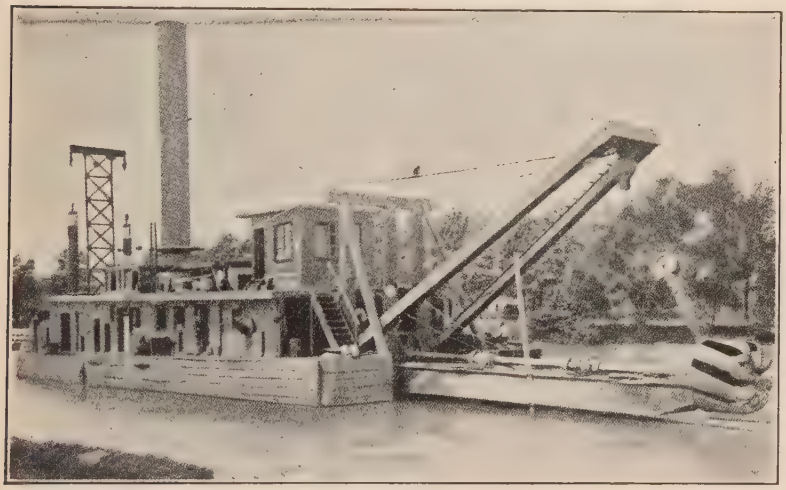


FIG. 153.—Hydraulic dredge.

which can make fills at any considerable distance from the place of excavation. A view of a hydraulic dredge is shown in Fig. 153.

The excavated material, consisting of mud, sand, gravel, or clay, is sucked from the bottom of the river by means of a centrifugal pump and transported through a pipe line to the shore and along the axis of the levee where it is discharged.

The suction pipe is attached to a frame or ladder, one end of which is hinged to the bow of the hull and the other end suspended from a boom. In some dredges, the ladder has a horizontal movement as well as a vertical one. The suction pipe on the ladder is connected to the stationary pipe on the dredge by either a vertical flanged joint or a universal joint. The end

of the suction pipe is surrounded by a cutting head which is attached to a shaft driven by an engine on the hinged end of the ladder. On the front end of the hull are mounted the boom, A-frame, and machinery for manipulating the ladder. A single-stage, high-pressure, centrifugal pump is located near the center of the hull on the longitudinal axis. The discharge pipe extends from the pump to the stern of the dredge where it is connected to the floating pipe, each section of which is supported by a pontoon, and the adjacent sections joined by a ball-and-socket connection. The forward end of each section of the shore pipe fits into the rear end of the next section and the sections are wired together, hooks being riveted to the ends for that purpose.

("*Bull.* 300," U. S. Dept. of Agriculture.)

FIG. 154.—Discharge pipe of hydraulic dredge showing material passing through openings in bottom of pipe.

The shore pipe is supported on trestles over the center line of the levee, as illustrated in Fig. 154. The last five to ten sections of the discharge pipe are constructed with bottom openings for discharging the water-borne material. These openings are about 4 by 6 inches in size and are from 8 to 10 feet apart. The openings are fitted with sliding gates to regulate the discharge.

Levees built by the hydraulic method generally have side slopes of 5 to 1. To assist in regulating the side slopes, slope boards are sometimes placed at the foot of the slopes or at intervals on the slopes.

Centrifugal pumps with discharge pipes ranging from 6 to 48 inches are used on hydraulic dredges, although for levee construction the 12-, 15-, or 20-inch size is ordinarily employed.

The percentage of solids carried by the water depends on the nature of the material and the velocity of discharge and varies from 5 per cent for coarse gravel to 30 per cent for fine sand and silt. Under average conditions, the percentage will be between 10 and 15. The capacities of pumps from 10 to 24 inches in excavating various kinds of materials are given in Table LXIV.

TABLE LXIV.—OPERATING DATA FOR UNITED STATES HYDRAULIC DREDGES[1]

Diameter of discharge pipe, inches	Material	Average gage pressure in discharge pipe, pounds per square inch	Average vacuum in suction pipe, inches	Total head, feet	Average output per hour of pumping, cubic yards	Coal used per cubic yard pumped, pounds	Total excavation, cubic yards
10	Sand	14	15	49	89	13.3	163,288
12	Sand and shells	9	20	43	97	15.63	9,172
12	Sand, gravel, shells	18	10	53	199	6.5	16,889
12	Sand	18	12	55	107	6.35	55,131
12	Sand and gravel	14	9.7	42	76	14.4	123,959
12	Sand and gravel	10	14	39	105	12.6	165,028
12	Mud	16	15	54	181	4.22	376,079
15	Mud, clay, sand	12.6	16.9	48	141	9.8	230,257
15	Gravel and sand	12	20	50	257	6.79	340,370
15	Silt, clay, and sand	22	18	71	172	8.64	488,875
15	Mud and silt	18	12	55	293	3.2	792,807
15	Mud, clay, and sand	3.5	11	21	309	2.71	1,078,285
15	Sand, mud, and shells	16	7	45	551	3.35	1,298,597
18	Sand and mud	14	18	53	113	15	228,263
18	Sand and mud, some rock	22	9	61	286	6.7	1,045,689
20	Sand and rock	15.4	16	54	336	5.27	365,433
20	Mud, sand, and shell	28	14	80	1,130	4.31	444,665
20	Sand, mud, and stiff clay	25	12	71	180	8.41	527,360
20	Mud and clay	28	18	85	685	2.45	636,417
20	Mud and sand	32	12	87	503	7.33	1,270,703
20	Mud and clay	21	15	65	827	2.48	1,341,835
20	Silt, quicksand	32	12	87	369	3.75	1,687,476
20	Mud, sand, clay	26	8	69	1,046	3.07	3,697,875
24	Sand and gravel	9	13.5	36	542	10.4	59,702

[1] *Annual Report*, Chief of Engineers, U. S. Army, Floating Plant (1915).

Hydraulic dredges are usually equipped with two spuds, both at the stern and about 10 feet apart. With one spud acting as a pivot, the dredge is swung in an arc by means of swinging lines fastened to the shore or to anchors. The raised spud is thus moved ahead of the acting spud. The forward spud is then

lowered and the other one raised, and the dredge swung in the opposite direction. In this way the dredge is gradually moved ahead, supplying the suction pipe with material.

Very substantial levees can be made by the hydraulic dredge at relatively low cost when suitable material in ample quantities is present in the river bed. The best material is clay with some sand. Obviously, it can be used economically only where the total yardage of excavation is large.

CHAPTER XV

DRAINAGE LAW

As generally considered, drainage laws are those laws which have been enacted by state legislatures expressly conferring rights of drainage upon a group of individuals in the interest of the public welfare. A very large and important part of the drainage law, however, is contained in that body of law known as the common law, which includes those principles, usages, and customs which grew into use by gradual adoption and received, from time to time, the sanction of the courts of justice, without any legislative act or interference. The principles of the common law are embodied in the decisions handed down by the courts.

In studying this subject, it is necessary to consider first of all the drainage rights of the individual under the common law before taking up the rights which have been conferred by statute upon groups of individuals; and in this chapter, the subject will be treated under these two classifications.

COMMON LAW OF NATURAL DRAINAGE

In some states the civil law of the Romans has been accepted by the courts as the common law of drainage; whereas in others, the common law of the English is upheld by the courts. Under the former law, the surface water flowing from higher lands, through natural depressions could not be dammed against by the owners of lower lands, while under the latter law the owner of the lower lands was allowed this right, surface water being considered a "common enemy."

Civil-law Rule.—The civil-law rule has been adopted by the states of Alabama, California, Georgia, Illinois, Iowa, Kentucky, Louisiana, Maryland, Michigan, North Carolina, Ohio, Pennsylvania, and Texas. The fundamental principle of this rule is that the owner of the higher land, known as the dominant owner, is entitled to the natural advantage which the elevation of his land gives him; and that the lower or servient owner must

receive the surface water flowing from the higher land through *natural depressions or swales.* The dominant owner also has the legal right, in the interest of good husbandry, to hasten the flow of the surface water by constructing artificial ditches on his own land, provided that the ditches do not cut through a natural divide and discharge water upon the servient heritage which otherwise would never have reached it. In addition, the dominant owner has the right to lay tile drains for removing the gravitational water in the soil, using natural or artificial ditches on his own land as outlets. The discharge from the higher lands, of course, must come through natural depressions, and the dominant owner cannot construct ditches and discharge the water onto the servient tenement at a point which is not the natural outlet of the basin.

Where a natural channel extends through the lands of both the dominant and the servient owners and, because of a choked condition of the channel or of insufficient slope, the water is not removed rapidly enough to suit the dominant owner, he may improve the channel on his own land but may not go onto the land of the servient owner to continue the improvement against the will of the latter.

Thus, if a landowner has adequate natural drainage under the civil-law rule, his land cannot be included against his will in a drainage district organized under the statute law.

Common-enemy Rule.—The English common-law rule, or common-enemy rule as it is generally referred to, has been adopted as the common law of drainage in Arkansas, Connecticut, Indiana, Kansas, Maine, Massachusetts, Minnesota, Missouri, Nebraska, New Hampshire, New Jersey, New Mexico, New York, Oklahoma, South Carolina, Virginia, Washington, and Wisconsin.

The fundamental principle of this rule is that surface water is a common enemy, and that a landowner may lawfully protect his land from the surface water flowing upon it from adjoining higher lands. Under this rule, the owner of high lands cannot improve natural channels or construct artificial channels on his own land, if by so doing he casts the surface water upon his neighbor to his injury, unless he secures an easement from his neighbor.

In the states following this common law rule of drainage, a drainage district organized for the purpose of providing better

outlets can assess all the lands in the watershed served by the ditches of the district.

Limitations.—As is clearly seen, the rules of common law apply only to those areas where there is a sufficient difference of elevation of the lands to cause natural drainage; and where the land is practically level, as is the condition of millions of acres of wet land in the United States, drainage cannot be effected by the individual landowner, since the common law does not permit one owner to cross the land of another in search of an outlet without the consent of the latter.

Recognizing the fact that it was to the public interest that these large areas of flat, fertile, wet lands be placed under cultivation, the legislatures of the various states have passed statute laws remedying the limitations of the common law and making it possible for the majority of the owners of adjoining lands to organize a drainage district for providing drainage outlets and to force the minority to join with them in the project. These laws were passed under the belief that it was against the best interests of the public to allow a few recalcitrant landowners to prevent or retard the economic and social development of a community. The provisions of the statute laws are enumerated and discussed in the following paragraphs.

STATUTE LAWS

Simple Drainage.—Under the common law, one landowner cannot construct drains across the lands of another without the latter's consent; but under the statute law, this is possible. A simple eminent domain proceeding before a Justice of the Peace, or other constituted authority, is generally all that is necessary. Of course the owner desiring drainage has to pay not only the cost of construction, but also any damages which will result from the work.

Mutual Districts.—The drainage laws of Illinois and of a number of other states provide for the organization of mutual drainage districts. Such a district is possible only when all the owners of the land to be drained are in full accord as to the plans of drainage and as to the apportionment of the cost. All that is necessary to form such a district is for the landowners to draw up and sign an agreement describing the work to be done and stating the division of the cost; and to have the agreement,

properly acknowledged, recorded in the drainage record of the county, township, or other political subdivision. Three commissioners to carry out the plans of drainage may be named in the agreement, or the court may be asked to appoint the commissioners. A district organized in this way has the same powers and the commissioners perform the same functions as provided for districts organized in a more formal manner. The advantages of mutual districts are that the organization costs are a minimum, and that much less time is required to perfect an organization. Because of the fact that it is difficult, if not impossible, for any considerable number of landowners to agree as to the best plan of drainage and an equitable division of the cost, mutual districts are generally small in area. Nevertheless, in the aggregate, a large amount of drainage has been accomplished in this way.

Majority-rule Districts.—This name is used to designate that large class of districts which have been organized under statute law by the coercion of the minority landowners by the majority. All large districts are organized in this way. The ensuing discussion of drainage under the statute law refers to majority-rule districts.

General Procedure.—Although the drainage laws of the various states differ in details, the general method of procedure is the same and is as follows:

1. Preparation of petition requesting the organization of a drainage district, and the signing of the same by the requisite number of landowners.
2. Filing of the petition with the proper authority.
3. Notification of property owners, by publication, of the filing of the petition and of the day and hour when the petition will be considered by the proper authority.
4. Hearing of the petition, when the form of the petition is examined to insure that the legal requirements have been met, and when objections as to the necessity of drainage are heard. If the petition is improperly drawn up or inadequately signed, or the objections as to necessity for drainage are sustained, the petition is dismissed; otherwise, the proceedings are continued and commissioners are appointed to examine the area to be drained, to prepare a plan of drainage, or to approve or modify the plan stated in the petition, and to determine the cost of construction and the amount of the benefits which will result from drainage.

5. Filing of commissioners' report.

6. Notice to landowners of hearing on commissioners' report.

7. Hearing on commissioners' report, at which time objections are heard as to the adequacy of the drainage plans and as to the relation between benefits and costs. If the court finds that the cost will exceed the benefits, the petition is denied; otherwise, provided the plans as proposed or as modified are acceptable to the court, the organization of the district is confirmed by the court. In this case the commissioners are ordered to prepare an assessment roll, showing the assessment against each tract of land and the damages which each landowner will suffer as the result of the drainage works. Usually, an appeal may be taken at this stage of the proceedings.

8. Preparation and filing of commissioners' assessment roll.

9. Notice to landowners of filing of assessment roll and time of hearing on same.

10. Hearing on assessment roll, at which time objections are heard by a jury only as to the amounts of individual assessments. If necessary, the jury views the lands of objectors and returns a verdict in the form of an assessment roll. The jury roll is confirmed by the court, and the assessments are turned over to the proper official for collection. Appeal may be taken by individuals on the ground that the assessment exceeds the benefit.

11. The commissioners issue bonds for defraying the construction costs. The bonds are a lien against the land within the district.

12 The commissioners let contracts and proceed to carry out the plans of drainage.

13. Provisions are made for maintaining the district, for enlarging the district, for constructing additional works, and for dissolving the organization.

The Petition. *Form and Contents.*—The definite form of the petition is not specified in any drainage law, and any customary form will answer. The promoters of a district should always secure the services of an attorney, versed in drainage law, not only to prepare the petition but also to advise them in regard to the details of organization and to represent them in court.

The contents of the petition are definitely specified in the drainage statutes of all states, and it is very important that all the legal requirements be carried out literally. At the first court hearing, the objectors may succeed in having the petition

denied, if they can show that it does not contain all the legal requirements or that some of these requirements have not been fully complied with. The contents of the petition vary somewhat in different states, but in general the petition must set forth:

1. The name of the proposed district.
2. The necessity for the same for agricultural or sanitary purposes, that is, that it will be conducive to the public health, safety, convenience, or welfare.
3. The boundary line of the proposed district.
4. The names of the landowners, when known, together with a description of the lands owned by each.
5. A description of the proposed starting points, routes, and termini of the proposed work.
6. A prayer for the organization of a drainage district, by the name and boundaries proposed, and for the appointment of commissioners under the statute.

Item 5 in the above list has always been a source of danger in the organization of a district. Before the "starting points, routes, and termini" can be determined intelligently in many instances, the services of a drainage engineer must be secured, and surveys and plans made. If the landowners promoting the district do not wish to obligate themselves to the extent of having such surveys made, insufficient data are at hand for giving a proper description of the necessary ditches and drains. Then, too, it is a mooted question in the courts as to how complete the description shall be. As a result of this condition, many petitions have been defeated on the grounds of insufficient description. The Missouri drainage law, applying to districts organized under the jurisdiction of the circuit courts, remedies this condition by omitting item 5. In Arkansas, under one system of organization, only two items are required in the petition: (1) a general description of the region to be embraced in the district; and (2) a bond to insure the payment of preliminary expenses. The more recent drainage laws also require a provision in the petition stating that the petitioners obligate themselves to pay all the costs of organization.

Requisite Signatures.—The number of signatures which must be affixed to the petition varies widely. In Iowa, Minnesota, and Arkansas (under one act), one signature is sufficient; in Florida under the Act of 1893, two or more owners are necessary; in Michigan, one-third of the landowners affected by the proposed

district must sign the petition; in Illinois, the petition must be signed either by the majority of the adult owners representing at least one-third the area or by one-third the adult owners, representing the majority of the acreage; in Missouri, under the county plan of organization, the petition must be signed either by the majority of the adult owners representing at least one-third of the area or by the adult owners of the major portion of the area; under the circuit plan of organization used in Missouri, the owners of the majority of the area must be back of the petition; while in Florida under the Act of 1913, a majority of either owners or acreage is required.

Thus it is seen that in some states one, two, or three landowners can get a petition before a court of competent jurisdiction; while in other states, a majority must be in favor of the project before the courts can assume jurisdiction. These two extremes are due to two lines of reasoning. Those favoring a few signers to a petition point out that since the court decides the question of organization in any event, based, of course, on the testimony of landowners, there is nothing gained by increasing the difficulties in getting the petition before the court, especially when the signers are willing to put up a bond for the payment of preliminary expenses in case the petition is denied. On the other hand, those favoring the signatures of a majority of owners or acreage argue that it is useless to bring the matter to court unless the majority are in favor of the undertaking. As would be expected, there are fewer fatalities under the latter plan. Attorneys and engineers are generally agreed that a middle ground would be better than either extreme used in existing laws, and that 10 to 20 per cent of landowners should be required to sign the petition.

As to who may sign a petition, all the laws simply state that a signer must be of legal age and own land within the proposed district. Nothing is said as to who shall sign when the land is an estate owned by tenants in common or joint tenants or an estate in which there is a life interest and a reversion or a remainder. There are no supreme court decisions on this subject, and the rulings of the lower courts have been so varied that the attorney for the petitioners has little to guide him in deciding whether his petition is adequately signed in a close case. As a result of this situation, petitions are frequently dismissed by the courts because of insufficient signatures.

Filing of Petition.—The petition is filed with the clerk of the court or tribunal by which it is to be heard. This may be either the circuit court, the county court, or the board of county or township supervisors. In most states, the petitioners can take their choice of courts. Small districts usually find it to their advantage to present the petition to the county or township supervisors, because of the smaller organization costs and the shorter time required to organize. Average size districts—which predominate in number—generally prefer the county courts; and large districts with land in several counties, the circuit courts. An important advantage of the county and circuit courts is that the records of proceedings are more carefully recorded and safeguarded.

Notice.—Within a stated time after the filing of a petition—usually 10 days or less—the clerk must notify all the landowners affected of the filing of the petition and of the time set for a hearing on it. This notice is given by the weekly publication of the petition in one or more newspapers of general circulation for a period of several weeks. In some states, printed notices are required to be posted at conspicuous points within the district. The form of the notice is usually prescribed by law.

Hearing on Petition.—At the hearing, the court decides just two questions: First, whether the petition contains all the legal requirements and is signed by the requisite majority; and second, whether the proposed works will be conducive to the public health, convenience, or welfare for agricultural or sanitary purposes. Objections may be filed with the clerk on, or before, the day of the hearing, or the objectors may be represented by an attorney at the time of the hearing. In most states, however, at this hearing, objections can be raised only on the grounds of the legality of the petition or of the necessity of the proposed drainage works. The judge hears all objections in a summary manner; if he decides against the petitioners on either point, the petition is dismissed at the cost of the petitioners; if the court sustains the petitioners, the proceedings are continued.

The procedure from this stage differs in the several states. Under the Missouri circuit court law, and those laws modeled after it, the court declares the drainage district a public corporation of the state, and a certified copy of the findings and decree of the court is filed in the office of the Secretary of State in the same manner as articles of incorporation are required under

the general law concerning corporations. Within 30 days after the incorporation of the district, and after all landowners have been notified by publication, a meeting is held for the election of five supervisors from among the landowners for carrying out the objects of incorporation.

In most states, the district is not formally organized at this first hearing, but the judge appoints three commissioners to have surveys, plans, and estimates made and to report to the court at a second hearing.

In those states where only a few signatures are required on the petition, as soon as the petition is filed, the court appoints an engineer to make the necessary surveys, plans, and estimates, so that when the first hearing is held there is a definite plan of drainage and an estimate of the cost for the landowners to consider. The majority of the acreage, however, must approve the project before the court can organize a district.

When the petition is addressed to the county or township supervisors, these officers act as drainage commissioners to investigate the necessity of drainage and to make plans of drainage.

No appeal can be taken from the court decision at the first hearing.

Preliminary Expenses.—This subject is of special interest to the engineer, since he is called on to give his services before the district becomes a corporate body legally capable of making a contract, and if the organization is finally defeated he has no redress from the district unless there is some provision in the law to meet the situation.

Under the Illinois law—except in the case of Special Drainage Districts under the Farm Drainage Act—and all the older laws, no provision is made for such expenses, and up to the point of confirmation of the assessment roll, the engineer, attorney, and others who serve the petitioners and commissioners do so at their own risk, since they cannot collect for their services under the law. The engineer can protect himself by making a contract with the petitioners as individuals.

In Alabama, Oklahoma, Arkansas, and other states where only a few signatures are required on the petition, a bond of $50 for each mile of proposed ditch is specified for the payment of preliminary expenses. In California, a bond in double the amount of the probable cost of the organization of the district is required by statute. Under the more recent laws of Mis-

souri, Arkansas, and Florida, a tax of 50 cents an acre is assessed against the land in the proposed district to cover the preliminary expenses. In Mississippi, the commissioners are empowered to borrow money on notes which are a lien against the land in the proposed district. In North Carolina, following the filing of the petition, the clerk of the Superior Court levies an acreage tax on the land represented by the petitioners sufficient to cover all the expenses incident to the organization of the district.

To protect himself, the engineer should be familiar with the laws of the states in which he practices.

Duties of Commissioners.—The commissioners appointed at the first hearing, or elected subsequently thereto, employ an engineer, if they desire, and proceed to carry out the duties of their office as prescribed by law. While these duties vary in minor details in the various states, in general they are as follows:

1. In case the petition contains a general description of the proposed work, the commissioners, with the assistance of the engineer, examine the area to be drained and determine whether the proposed location of the ditches, drains, levees, and other works is proper and feasible and, if not, modify the proposed plan so as to obtain the most feasible plan of drainage. Where no plans are contained in the petition, the commissioners proceed to prepare the plan of reclamation.

2. They determine whether the district as proposed in the petition contains all the lands which will be benefited by the improvement, and whether lands are included in the district which will not be benefited. In the former instance they include such lands in the district; in the latter, they exclude the lands not benefited.

3. They determine the probable cost of the construction of the proposed work and whether such cost will be less than the aggregate amount of the benefits to be derived.

4. They determine what lands will be damaged by the proposed work and the probable aggregate amount of all damages.

The report of the commissioners is filed with the clerk of the court who proceeds to notify the landowners by publication of the time set for the hearing on the commissioners' report.

Second Court Hearing.—Under the circuit court plan of Missouri, Arkansas, and Florida, the judge appoints three commissioners to assess benefits and damages under the plan of drainage recommended by the supervisors.

In most of the states, at the second hearing, objections may be raised as to: (1) the plan of the commissioners; and (2) the proportion of benefits to cost. These objections are heard in a summary manner by the court, desirable modifications are made in the plans, and the district is formally organized, if the judge is convinced that the plan is good and that the benefits will exceed the cost. The commissioners are ordered to prepare an assessment roll of benefits and damages against each tract of land. An appeal may be taken from the decision of the court.

Preparation of Assessment Roll.—The commissioners proceed to view the land in the district and to estimate the amount of benefits and damages which will accrue to each 40-acre tract of land, or other subdivision according to ownership, as the result of the proposed drainage works. The levying of an equitable assessment of benefits against individual tracts of land is one of the most important and the most difficult steps in the organization of a district. This subject is of such importance that the methods of making drainage assessments will be discussed in detail later in the chapter. The commissioners agree with the landowners, as far as possible, as to the cost of right of way for the ditches, holding basins, and other necessary structures.

The findings of the commissioners are arranged in tabular form, the columns of which are generally prescribed by law as follows: column one, "Owner of property assessed;" column two, "Description of property assessed;" column three, "Number of acres assessed;" column four, "Amount of benefits assessed;" column five, "Number of acres taken for right of way;" column six, "Value of property taken;" column seven, "Damages." The total amount of the assessments, less the amount of damages, must equal the estimated cost of construction and organization. This report of the commissioners is known as the "commissioners' assessment roll," and is filed with the clerk of the court.

Hearing on Commissioners' Assessment Roll.—After all the landowners concerned have been properly notified, a hearing is held on the commissioners' assessment roll. A jury is impaneled to decide all questions of benefits and damages to any land in the district. Any landowner who objects to the amount assessed against him may present his case to the jury in person or by counsel and offer any competent evidence in regard thereto. The jury, after listening to the evidence presented and, if neces-

sary, viewing the lands under discussion, may make any modification in the assessment roll which appears just, and return its verdict to the court in tabular form similar to the commissioners' assessment roll. This is known as the "jury assessment roll" and shows the benefits which are expected to accrue to the lands, railroads, highways, and streets affected by the improvement, and the damages to the lands taken or damaged by the construction of the proposed works.

The court confirms the jury roll and causes the assessments to be entered upon the records; and this judgment and verdict is a lien upon the land until the assessment is paid. Individual appeals and writs of error are allowed from the verdict of the jury.

Method of Paying Assessment.—Any landowner who so desires may pay the full amount of his assessment in one payment; but generally the assessments are paid in 10 or 15 annual instalments with interest on deferred payments. In some states, the first instalment may be deferred for several years to permit the annual instalment to be paid from the increased revenue from the improved land.

Bonds.—To pay for the construction work, the commissioners are empowered to issue bonds in an amount not exceeding 90 per cent—in most states—of the amount of the unpaid instalments against the land, maturing at annual intervals, and running not more than 1 year after the last instalment is due. The minimum price at which the bonds shall be sold is also generally specified by law; in Missouri, the minimum price is 95 per cent of the par value and accrued interest.

Contracts for Construction.—The commissioners have the power to make contracts in the corporate name of the district, to be contracted with, to sue and be sued, and to do all the acts necessary for carrying out the plan of drainage.

After the assessments have been paid or the bonds sold, the commissioners enter into contracts for the construction of the drainage works. All contracts over a certain sum—usually $500—must be let to the lowest responsible bidder, after proper advertisement for bids, as in other public work, the right being reserved to reject any and all bids.

Additional Assessments.—Provision is made for additional assessments in case the cost of construction exceeds the engineer's estimate and the amount of the original assessment. In

some states, an additional assessment may be spread by the court upon petition of the commissioners; in others, proceedings similar to the original proceedings must be followed. In no case, shall the sum of the assessments exceed the benefits assessed against the land.

Provision is also made for maintenance work. The commissioners report each year to the court upon the work and expenditures of the past year and upon the needs for the coming year. The court allows the amount asked for maintenance if in its judgment the work is necessary and the amount is reasonable. The maintenance tax is usually so much an acre, and the amount in any one year is limited by statute; in Illinois, the annual maintenance tax is limited to 30 cents an acre; in Missouri, the maintenance tax is apportioned upon the basis of the net assessments of benefits accruing for original construction and is limited in any one year to 10 per cent of that amount.

Other Provisions.—Provision is made in the statute law for annexing lands to the district as originally organized, and the procedure is specified and the method of assessing new lands; also, for the formation of subdistricts for providing more minute drainage than given by the original work of the district.

Special Legislation.—Practically all of the drainage work which has been done has been effected under the common law of drainage or under some general statute law similar to the one described. Many states have two or more independent drainage laws to meet more nearly the varying conditions of size of district, political location of district, and nature and completeness of drainage desired. In some states, notably Arkansas, California, Florida, and North Carolina, drainage districts have been organized by direct legislation; for instance, the Sacramento Drainage District—embracing lands in 10 counties along the Sacramento River—which was created by special enactment of the California Legislature in 1905. Arkansas has passed a number of such acts, as many as six in the 1917 session of the legislature. California has passed two, Florida one, and North Carolina one.

Management of Districts.—In most states, the management of drainage districts is entirely under the control of the landowners through their elected commissioners, subject only to certain statutory limitations. In other states, the organization and management of districts are subject to the approval of

some state or county official; and in some cases, the organization is initiated by such officials. This feature of state control is found in the laws of California, Louisiana, Nebraska, Massachusetts, Minnesota, Montana, Oregon, Utah, and Wisconsin.

State Aid.—It is beginning to be recognized that land reclamation by drainage is of public as well as private benefit. If drainage is considered of private benefit only, such enterprises cannot be aided by public moneys or vested with the power of general taxation. There are several supreme court rulings on this point.

A recent decision[1] in Oregon states in part:

> Drainage for the benefit of the public health is for a public use, and drainage for the reclamation of large tracts of swampy lands for agricultural purposes is likewise considered to be for the public benefit.

The United States Supreme Court in the case of Fallbrook Irrigation District[2] vs. Bradley said:

> To irrigate and thus bring into possible cultivation these large masses of otherwise worthless lands would seem to be a public purpose and a matter of public interest, not confined to the landowner, or even to any one section of the State.

The Minnesota Supreme Court has repeatedly emphasized the public character of the reclamation of wet and overflowed lands in large areas. In one case[3] the Court said:

> That the drainage of large tracts of wet and overflowed lands will operate beneficially to the public, there can be no serious doubt . . . While the element of public health is often made an important factor in the consideration of statutes of this kind, it is believed that any public benefit, such as the improvement of highways or the reclamation of large tracts of otherwise waste lands, is sufficient to support and sustain them.

Comparatively few states have made provision for direct financial aid, and among these there has been no uniformity of practice. In some instances it takes the form of a direct legislative appropriation, and in other instances the county courts are authorized to appropriate county funds. Sometimes it is a case of outright gift and other times it is only a case of lending with provision for repayment. The practice in the several states will be given briefly.

[1] *Drainage District No. 7 v. Bernards*, 174 *Pac.* 1167.
[2] *Fallbrook Irrigation District v. Bradley*, 164 *U. S.*, 112.
[3] *Utick v. Commissioners of Polk Co.*, 87 *Minn.*, 325; 92 *N. W.* 216, 1902.

Alabama.—In 1919 an act was passed to promote the public health, convenience, and welfare by leveeing, ditching, and draining wet and overflowed lands. The court or county commissioners are authorized to appropriate funds to aid in the construction and maintenance of such improvements. No limit is set by the Act and no provision is made for repayment. Other sections of the Act, however, indicate that the primary burden is to be borne by the lands benefited.

California.—The Act of 1905, creating the Sacramento Drainage District, provides that a committee of three of the nine drainage commissioners of the district, acting in conjunction with a committee of three appointed by the Governor, should determine the proportion of the expense to be borne by the district and by the state, respectively. The Act also provides that the amounts assessed against the lands in the district shall not be payable until the legislature shall have made an appropriation or other legal provision for the payment of its share. The Act was declared constitutional in the case of *People v. Sacramento Drainage District.*

Kansas.—In 1917 provision was made for a State Water Commission, consisting of the Governor of the State, ex-officio chairman, and two engineers appointed by him. The duty of this commission, in conjunction with the Federal Government by way of securing financial and professional aid, is to work out a systematic general plan for the complete development of each watershed in the State in order to secure the most advantageous adjustment of the interests involved in matters of floods, drainage, irrigation, water power, and navigation. The commission establishes and maintains river gaging stations, and makes such surveys and other investigations as are necessary.

Kentucky.—A county may remove any ponds, pools, or swamps, or reclaim swamp land that may cause sickness, and may pay for the same out of the county levy or by taxation of the property in the county subject to taxation for state purposes.

Minnesota.—Minnesota has in several instances appropriated money for the deepening, widening, straightening, diking, and otherwise improving important streams for flood control purposes. The Act of 1909 relative to the Big Stone Lake and Whetstone and Minnesota rivers is an example. Also, an Act was passed in 1913 authorizing the State Drainage Commission to construct an outlet for the waters of the Mustinka state ditch

in Traverse County, and appropriating money therefor; a similar act was passed relative to the Snake River in Marshall County.

In 1919, a Department of Drainage and Waters was created with a commissioner at its head who is appointed by the Governor. The Commissioner of Drainage and Waters is authorized to cooperate with county boards or with landowners in certain cases to the extent of 50 per cent of the cost, where the project involves the deepening, widening, and straightening of natural water courses. And county boards are likewise authorized to appropriate money of the county revenue fund to aid in such projects. The Commissioner of Drainage and Waters is also authorized to make all necessary investigations to ascertain and determine the topographic features of the various watersheds of the State, and to determine the works necessary to secure proper drainage outlet for the lands in each basin and the construction of the necessary works to improve such outlet and to control the flood waters therein. He is also to determine the probable run-off in each important watershed in the State. He is empowered to supervise the organization and construction of all drainage districts.

North Dakota.—In 1919, North Dakota created a flood control commission, consisting of the dean of the state school of mines, the state engineer, and two citizens appointed by the Governor, to make surveys and plans for controlling floods.

Ohio.—By the Act of 1919, the Superintendent of Public Works is made ex-officio drainage commissioner. His duties are to carry out the provisions of the law relating to cooperation with local, state, or federal authorities in the construction, improvement, or altering of any stream, river, or ditch for drainage purposes. He makes inquiry into the need for the improvement of rivers or ditches used as general outlets for the purpose of reclaiming land, and offers such assistance as may be necessary to the proper procedure of local or state authorities.

When one or more boards of county commissioners are petitioners for an improvement that will materially increase the value of the lands affected thereby for general taxation, they may apply to the Superintendent of Public Works for state aid. If the superintendent is convinced of the increased values and that the improvement will be beneficial to the counties and the state, he may grant such per cent of the estimated cost as in his judgment will be commensurate to the advantage that

will accrue to the state in 10 years, but not in any case to exceed 25 per cent of the cost; but provided that the county or counties affected shall give at least one-half the amount given by the state. The general tax levied on the *increased* value of the land is paid into the public treasury from year to year until the state has been fully repaid with 4 per cent interest. Counties may participate to the extent of 10 per cent of the cost of drainage projects.

Texas.—Texas has a State Reclamation Department whose duties are to make surveys, investigations, and plans necessary for the reclamation of the swamp and overflowed lands within the state. It has supervision over all drainage work done; but the state pays no part of the cost of reclamation.

Wisconsin.—The drainage laws of Wisconsin, passed in 1919, require that the preliminary report of the commissioners of a drainage district must include a report from the state chief engineer regarding the practicability and completeness of the proposed plans of improvement, and a report from the College of Agriculture of the University of Wisconsin regarding the character of the soils and the value of the land for agriculture. If the enterprise will affect any navigable stream the plan must be approved by the state railroad commission.

Illinois.—The Illinois Legislature passed an Act in 1921 requiring that any drainage improvement made in the state which will affect the flow of water in any stream must secure the approval of the Division of Waterways.

Special and General Taxation.—The usual method of raising funds for drainage improvements is by special assessment on the land benefited, proportional to the benefits received. General taxation as a means of financing drainage projects is the exception, and not the rule.

In Arkansas, the Legislature of 1917 created a number of drainage districts by special act, and provided for the levy of both a percentage and an acreage tax.

In California, the Los Angeles County Flood Control District was created by special legislative enactment, and provision was made for a general tax.

In Kansas, by an act passed in 1911, provision is made for the levy of both general taxes and special assessments. The general tax covers those general benefits which will accrue to the district as a whole from the improvement, and is at an

equal rate on the assessed value of all the lands in the district. The special assessment is based on special benefits in proportion to the benefits accruing to the particular tract.

Missouri, Arkansas, and Florida levy a general tax to cover the preliminary expenses of organization.

Texas and Louisiana provide by separate laws for the raising of funds either by general taxation or by special assessment. The general tax provision applies to districts which have issued bonds, and provides for an ad valorem tax upon all the property in the district, both real and personal, sufficient to retire the bonds as they mature.

Probably a combination of a general tax and a special assessment is the most equitable arrangement for the landowner. The advantage of this arrangement and the disadvantage of the present one are well stated by Harman:[1]

> When a tax is levied on a benefit basis in a drainage or levee district the lowest lands, probably remote from highways and so located as to be the last which can be developed and put into cultivation, will frequently receive the highest tax and may be called upon to pay the interest and annual instalment of taxes for from 2 to 5 years before the land can be put into cultivation. Furthermore, this land, being the lowest, carries the greatest hazard in case of partial failure of the levees to protect from overflow or the ditches to give effective drainage. Under such partial failure, this low land may become the temporary storage reservoir for draining adjacent lands of more favorable elevation.
>
> If a portion of this burden of taxation is placed upon the district on an ad valorem tax basis the lands of greater value and utility will have the greater part of this portion of the tax. Then, when the lands less favorably situated are made available for cultivation, the annual assessment for general, state, and county taxes, and consequently for annual drainage taxes, will increase with the value and availability of the land. In that manner such land will pay its due proportion of the "Cost of Reclamation."

ASSESSMENTS FOR DRAINAGE DISTRICTS

As previously stated, the preparation of the assessment roll by the commissioners is the most difficult step in the organization of a drainage district. The critical stage in the organization proceedings is reached when each landowner finds out

[1] HARMAN, JACOB A., "A Proposed Uniform Law for Land Reclamation by Drainage," *Eng. News-Record*, Apr. 27, 1922.

definitely the amount assessed against his land. Appeals of objectors from the judgment of the court often delay the construction work several years, and in some instances result in the abandonment of the organization. Objectors are often made not so much by the amount of their assessments as by the inequitable apportionment of the cost; hence, it is of the utmost importance that the basis of assessment should be equitable and simple of explanation and application.

The laws of some states require that the engineer of the district shall be a member of the assessing board; in other states, the engineer simply advises and assists the commissioners in making the assessments. It is, therefore, essential that the engineer should be fully informed as to the laws of the states in which he practices in regard to the method of making assessments, and that he be familiar with the best and most equitable method of carrying out the provisions of the law.

Principles of Special Assessments.—The customary method of financing drainage projects is by special assessment on the land benefited although, as previously mentioned, in a few instances part of the cost has been defrayed by general taxation on an ad valorem basis.

A special assessment is not a tax, as that term is used in the state constitutions, but may be defined as an enforced charge upon real estate, imposed by competent political authority, to secure funds to pay for an improvement of a public character whereby the real estate receives a special benefit, the amount of the charge not exceeding, and apportioned according to the amount of such special benefits. The courts have universally upheld the right of legislatures to make laws levying special assessments upon property for local improvements of public benefit. Thus it is that all drainage laws contain a provision to the effect that the proposed improvement must be "conducive to the public health, convenience, or welfare, or of public utility or benefit," and also a provision conferring upon a district the right of eminent domain.

The fundamental principle of special assessments is that the assessments must not exceed the benefits, and that they must be apportioned according to the benefits.

Special assessments may be levied against agricultural lands, railroads, highways, counties, municipalities, and other corporations if they receive benefits from the improvement.

Definition of Benefits.—Since special assessments are based on benefits, it is necessary to understand what is included under that term. Benefits are of two kinds, general and special, both of which must be present in any local improvement, but only the latter can be assessed.

General benefits are those which are received by the community at large, such as better health, better roads, and greater general prosperity.

Special benefits are those which will increase the value of the land by making it either more productive or more desirable or better adapted for the use which is made of it. Special benefits may be either direct or indirect, but they must be reasonably certain and must develop within a reasonable time. Special benefits, also, must be the result of the improvement itself, and not the result of some future additional improvement or of some natural advantage inherent to the land, such as highness of land, natural drainage, and fertility and structure of the soil—advantages which the owner paid for when he bought the land.

Benefits to Agricultural Lands.—In determining the benefits which will accrue to agricultural lands, the several factors to be considered are: (1) wetness of land or need of drainage; (2) completeness of drainage furnished by the improvement; (3) increased accessibility; (4) fertility of soil; (5) condition of land for cultivation; and (6) extraordinary benefits.

Wetness of Land.—In most proposed drainage districts, lands of varying degrees of wetness are found. It is not so much the actual amount of water upon the surface or in the soil that is the important factor as it is the handicap which the water places on the land when considered for agricultural purposes. Thus, some lands are high enough to produce fair crops in years of ordinary rainfall; others can produce only shallow-rooted crops such as hay; and still other lands are entirely unproductive. This is the basis upon which benefits should be assessed instead of some arbitrary classification of lands, such as "high," "low," "wet," and "swamp." When considered as to wetness and need of drainage, the unproductive lands will receive a greater benefit from drainage than those which are partly productive and, therefore, should be given a higher assessment, other factors being the same. There is no arbitrary relation, however, between the wetness of the several classes of land which can be used in all cases, although such a fixed relation is common practice in some

states; but such relation must be determined independently for each district. Allowance must be made for the natural advantage of highness of lands, especially in those states following the civil-law rule of common-law drainage, where the owner has the legal right to cast the water from his land upon the lower adjoining lands.

Completeness of Drainage.—While the wetness factor is ordinarily the most important, the completeness-of-drainage factor is frequently of equal or even greater importance. Too often, the commissioners assume that the proposed ditches and drains will drain all the land completely and make the assessments on that basis. But this condition is extremely rare, and it is manifestly unfair to assess lands for benefits which they will not receive. The completeness of drainage will depend on the elevation of the land, the proximity to the ditch or drain, the adequacy of the outlet provided, and that of the ditches themselves.

Some of the land in the district may be so low that the ditches or drains will not lower the water table sufficiently to drain the land properly, and this fact must be taken into account in making the assessment. On the other hand, if the ditches of the district are made much deeper than otherwise necessary to provide a satisfactory outlet for a tract of very low land, such land, being at a natural disadvantage, will receive greater benefits than the more favorably situated lands, and should be assessed accordingly.

The lands nearest the ditches will receive the most complete drainage. Lands at a distance from the ditches are at a disadvantage, since to receive adequate drainage and to use the ditch as an outlet, lateral drains must be constructed at individual expense. The cost of such additional drains should be deducted from the total benefits received by the land on the assumption of complete drainage. This cost is not dependent solely upon the distance of the lands from the ditch—as is the practice in a number of states—but also upon the kind of drain which will be necessary, its slope, and its size. Thus, the allowance to be made for lands at a distance from the ditches or drains provided in the drainage plan will vary in different parts of the same district for lands equally distance from the drains, and should be considered separately for each tract of land.

In those states following the common-enemy rule of common-law drainage, distance from an outlet is considered a natural disadvantage, and such lands are considered to receive greater

benefits from the improvement than the lands along the ditches, provided the ditches and drains are located in marked natural depressions, rather than located more or less arbitrarily by the engineer and commissioners. In the latter instance—which generally applies to the lateral ditches and drains—distance of land from an outlet decreases the benefit from the improvement.

Occasionally, the outlet stream is inadequate to remove the water brought to it by the main ditch of the district, and the low lands, especially at the lower end of the district, are not only not benefited by the improvement, but are even damaged by it. In making plans for the district, the engineer should see that a sufficient outlet is provided. Frequently this means widening and deepening the outlet stream for a greater or less distance below the end of the district. If this is not possible, however, and if, as a result, the low lands will not receive complete drainage or will be damaged by the proposed work, the commissioners should take this fact into consideration in making out the assessment of benefits and damages.

After the district is organized, the owners of lands damaged by flooding cannot sue the district, but they can institute mandamus proceedings against the commissioners and compel them to provide adequate ditches and outlets.

It is seldom that the ditches of a district are designed to handle the run-off from exceptional storms which occur on an average of once in 10 to 20 years. When the ditches overflow their banks, it is the originally wet and swamp lands which are damaged —the very lands which are generally assessed the highest on account of their need of drainage. Knowing that the lower lands will be damaged periodically from flooding, the commissioners should make allowance for such damages by reducing the amount of benefits assessed against these lands.

Increased Accessibility.—Some lands in a district may be so high as not to need artificial drainage, but may be surrounded by wet and swamp lands or separated from the body of the farm by a swale or slough which makes access to the high land more or less difficult or inconvenient. The courts have held that increased accessibility is a benefit, and that high lands rendered more accessible by the drainage of adjacent low lands can be assessed to the extent of this benefit.

Fertility of Soil.—In Delaware, Georgia, Kentucky, Mississippi, North Carolina, South Carolina, and Virginia, the statute

includes fertility of soil as one of the factors which must be considered in making drainage assessments; and in most of the other states it is usually inferred. In using this factor considerable caution must be exercised.

On the one hand, fertility of soil is a natural advantage of the land, an advantage which the owner acquired when he bought the land. The fertility of the soil is not increased by drainage and, therefore, should not be considered in assessing the land for drainage improvements.

On the other hand, if fertility of the soil is not considered, and the cost of the improvement is apportioned according to the need and completeness of drainage, poor land will be assessed too large a share of the total cost, when the financial returns from the cultivation of the land is considered.

Since the basis of assessments for drainage, however, should be the benefit to the land expressed as the increased market value of the land, and since this increase will always be greater for rich than for poor lands, it seems only just that the fertility factor should be included; but the fact that fertile, undrained land is more valuable than poor, undrained land must not be overlooked, and the owners of the fertile lands must not be made to pay a second time for a value which they already possess.

Condition of Land for Cultivation.—The lands in most proposed districts are in different stages of preparation for cultivation. The higher lands may be under cultivation, other lands not under cultivation may have been cleared of trees and stumps, and still other lands may be covered with trees. Frequently, commissioners place a higher assessment on cleared than on uncleared lands on the grounds that cleared land can be placed under cultivation at less expense and sooner, and, therefore, will produce revenue before the uncleared lands. This is not a just basis of assessment, since it penalizes the foresighted, progressive landowner who at his own expense has placed his land in condition to receive immediate benefits from anticipated drainage. Other factors being the same, cleared and uncleared lands should be assessed equal amounts, and the owner whose land is in condition to receive immediate returns from the improvement is entitled to them because of his larger investment and his foresightedness.

Extraordinary Benefits.—Extraordinary benefits are those which certain lands receive by virtue of the location of the drain-

age system. The owners of lands, for example, through which a tile drain or ditch lateral is located, receive an extra benefit due to the location of the drain, since they are thus provided with an outlet for their tile laterals at the expense of the district, a benefit which is lacking to other lands. The amount of this benefit is generally taken as the cost of a tile submain of sufficient size to receive the discharge of the tile laterals on the individual tracts.

Methods of Assessing Agricultural Lands.—The drainage statutes of all states provide that the total cost of the drainage works shall be borne by the lands in proportion to the benefits received, and that in no instance shall the assessment exceed the benefit. The courts have universally held that the increased value of the land, as the result of the proposed work, is the correct measure of the benefits received by the land. Fourteen states require that the benefits to each tract of land shall be expressed on the assessment roll in dollars and cents, and shall represent the estimated increase in the value of the land. Seven states require that the lands shall be divided into five classes and that the scale of assessment upon the several classes shall be in the ratio of 5, 4, 3, 2, and 1, respectively. Other states do not specify the method of making the assessment, and various methods are used. Regardless of the method used, the courts have generally upheld the assessment roll, provided the individual assessments appear just, and not in excess of the benefits.

The best time to stop litigation is before it develops; this can be done to a great extent by employing a method of making assessments which is reasonable, simple, and easily understood. In addition, the commissioners should explain their method to all interested landowners and should invite the landowners to accompany them when viewing their lands and to give any information having a bearing on the subject. In this way, many objectors can be converted into friends.

The various methods used in arriving at the amount of the individual assessments will be briefly considered.

Increase in Value Method.—This method consists in estimating the increase in value which will accrue to each tract of land from the proposed plan of drainage, and assessing such proportional amount of this increase in value as will make the sum of all the assessments equal to the cost of the improvement. In determining this increase in value, the present value of each tract of land

is subtracted from the estimated value of the land after the improvement is completed.

The value of the undrained land may be based either on its fair market value or on the revenue which it produces. The fertility of the soil, the condition of the land for cultivation, and the wetness of the land should be considered in estimating the present value. There is no definite rule to follow, and the fairness of the values arrived at will depend almost entirely upon the good judgment of the assessors and upon their knowledge of local values.

The future value is more difficult to estimate since there are more factors involved. First, the value of each tract, completely drained, is given a value based on the market value of adjacent high lands which already have complete drainage, taking into account any difference in fertility of soils. The difference between the value of the undrained and drained land is the amount of benefits to the land on the assumption of complete drainage. But all the lands will not be completely drained, and deductions must be made for low lands which will be only partially drained and which will be subject to periodic flooding, and for those lands at a distance from the ditches which will require additional drains, constructed at private expense, to complete their drainage. Also, additions must be made in the case of those lands which will receive extra benefits.

In the author's opinion, this is the best method of making drainage assessments and should be used where it is not in direct conflict with the statutes. Its advantages are that it follows directly the theory of local assessments, is simple and readily understood by both assessors and landowners, and gives the landowner definite information on the amount of his assessment at the final hearing in time for him to file objections if he so desires. Also, the sale of bonds is more readily and advantageously effected, since the bond houses have fairly definite information as to the security back of the bonds.

The Classification Method.—Under this method, the land is divided into five classes, designated A, B, C, D, and E, and the ratio of assessment between the classes is 5, 4, 3, 2, and 1. The land receiving the most benefit is placed in class A, that receiving the least benefit is placed in class E, and the rest of the area in either B, C, or D. In classifying the lands, the wetness of the land, its proximity to the ditch or natural outlet, and the fertility of the soil are the factors to be considered as specified in the

statutes. This method has many disadvantages, chief of which is that the landowner does not know the amount of his assessment in dollars and cents until after the classification has been confirmed by the court and it is too late for him to offer any objections. The only recourse he has is an appeal from the decision of the court. Further, this method lacks elasticity; it is extremely difficult, if not impossible, to divide the lands into five classes, so that the benefits will be in proportion to the ratio fixed by law.

The Percentage Method.—Several states, notably Iowa and Illinois, under the Farm Drainage Act, require that the commissioners shall make special assessments for benefits by classifying the lands in the district in tracts of 40 acres, more or less, according to the legal or recognized subdivisions on a graduated scale, to be numbered according to the benefits to be received by the contemplated drainage. The tracts of land which will receive the most and about equal benefit shall be marked 100, and such as are adjudged to receive less benefit shall be marked with a smaller number, denoting the per cent of benefit. This method is somewhat similar to the classification method, but is more elastic, since the number of classes is unlimited and the ratios of benefits vary from 1 to 100 instead of from 1 to 5; consequently, it is possible to apportion the cost more equitably than in the former method. The objections to this method are that it is difficult for both the assessors and the landowners to express benefits by percentages, and that the landowner does not know the actual amount of his assessment—the only basis upon which he can judge the value of the improvement to him—until after the classification has been confirmed by the court and it is too late for him to enter any objections.

The usual method of applying the percentage method is as follows:

1. The several factors of benefit such as need of drainage, completeness of drainage, proximity to the drain, distance from a natural outlet, and fertility of soil are considered separately and each tract is rated on a percentage basis under each of these classifications.

2. The several per cents for each tract are then multiplied, and the products are added.

3. The extraordinary benefits which will accrue to the individual tracts are estimated in dollars and cents and totaled.

4. The sum of total extraordinary benefits is subtracted from the total estimated cost of the work.

5. The remaining cost is then divided by the sum of the products obtained in step 1, the quotient being the monetary value of 1 per cent unit of benefit.

6. This unit of benefit is multiplied by the number of units for each tract obtained in step 2, and the resulting products are multiplied by the number of acres in the tracts, respectively.

7. The amount of any extraordinary benefit found in step 3 is added to the amount determined in step 6, and the sum is the actual assessment against each tract of land.

8. The amounts of the assessments obtained in step 7 are then reduced to a percentage basis. The tract receiving the largest benefit assessment is marked 100, and the other tracts are marked in proportion thereto. These percentages are then listed in the classification roll and filed with the court.

As used in practice, this method has not given satisfactory results. In an investigation of drainage assessments made by the Agricultural Engineering Division of the United States Bureau of Public Roads,[1] it was found that in most states where this method is used it is customary to divide the lands into four classes with respect to need of drainage, and to rate them 100, 70, 30, and 5 per cent, respectively. It is evident that, while this ratio might have been correct in some district, it cannot apply to all districts; and commissioners should not use these ratios just because they were used satisfactorily in some other district.

Likewise, it was found that the proximity-to-ditch factor was generally established on the following basis:

	Per Cent
Lands through which the ditches or drains pass	100
Lands lying ¼ mile from an outlet	75
Lands lying ½ mile from an outlet	50
Lands lying ¾ mile from an outlet	25
Lands lying 1 mile from an outlet	5

The use of such ratios as these cannot give satisfactory results. As previously pointed out, the disadvantage which a tract of land suffers by not having a direct outlet cannot be expressed fairly simply by its distance from an outlet. This disadvantage

[1] *Bull.* 1207, United States Department of Agriculture.

should be measured by the cost of a drain from the land under consideration to the drains of the district; and this cost may, and will, vary considerably for lands at equal distances from an outlet.

If correctly used, this method may give fair results, although it is cumbersome and indirect. To do this all the natural, artificial, and legal advantages of each tract must be considered. Also, the assessors must be accustomed to thinking in percentages, a faculty which is rarely found in drainage commissioners.

The Flat-rate Method.—As its name implies, this method consists in assessing an equal amount against each acre in the district. Obviously this method is rarely applicable to drainage districts. It has been used mainly in levee districts where equal protection against overflow is afforded to all the lands. It applies also to districts which are organized for straightening, widening, or otherwise improving the channels of natural streams, thereby protecting the lands from overflow. This method has the advantage of eliminating inequality of assessments—a factor which is frequently the source of litigation.

Arbitrary Assessment Method.—This is probably the oldest method of making assessments, but it is rarely used now. Under it, the total cost was divided by the number of acres within the district. This average cost was the assessment against most of the land. The assessment against the wetter lands was increased arbitrarily as seemed just in the opinion of the assessors; and the assessment against the higher and drier lands was decreased in the same manner.

Damages to Agricultural Lands.—The commissioners are required not only to assess benefits against the land, but also to determine the amount of any damages which will result from the proposed drainage work. Benefits and damages must be determined independently, and in many states damages cannot be offset against benefits, but must be paid before the construction work starts.

The principal element of damage is that caused by the taking of land for right of way for ditches, holding basins, and pumping plants. The courts have held that damages cannot be obtained for the construction of tile drains or for an open ditch located in a natural channel. The measure of damage to lands taken by the district is the value of the unimproved land. The final decision as to damages rests with a jury, as in other condemnation proceedings.

Damages can also be claimed for inconvenience caused by the cutting off of a portion of a tract of land by an open ditch, making the separated portion difficult of access or difficult of cultivation on account of its size and shape. The question of access is generally settled by the construction of a bridge at the expense of the district. Damages due to the second cause will vary with conditions, and constitute a matter to be arranged between the commissioners and the landowner; and in case they cannot agree, the amount of damage is fixed by a jury. It has been held generally that the measure of damages is the difference in the value of the land before and after the improvement.

Assessments against Railroads.—The drainage laws of all states provide for the assessment of railroads within a drainage or levee district. The laws of some states specify what benefits shall be considered in assessing railroad rights of way, whereas others simply state that the assessment shall be in proportion to the benefits derived from the proposed improvement. In some states, the statutes recognize the difference in the nature of the benefits accruing to railroads and those accruing to agricultural lands; and the courts of nearly all the states agree on this point. The Supreme Court of Minnesota has held that railroads which pay a gross-income tax to the state cannot be assessed for drainage-district improvements.

Unquestionably there has been more litigation over railroad assessments than over those for agricultural lands, and this has been due almost entirely to the illogical basis upon which railroads have been assessed.

Basis of Assessment.—According to the theory of special assessments, as interpreted by the courts, property must be increased in value, relieved of some burden, or made better adapted to the purpose for which it is used, before it can be assessed.

The basis of increase in value used for agricultural lands should not apply to railroad rights of way, since the land used by the railroad is not used for agricultural purposes and could rarely be economically used for such purposes, even if the land should eventually revert to the owners of adjacent lands; and further, the market value of a railroad, as a whole, is not affected in the slightest degree by the drainage afforded by a drainage district, even though admittedly benefited thereby. Hence, railroads should not be assessed on an acreage basis, but on a lump-sum basis, the amount representing the equalized value

of the benefits due to the relief from some burden or to the betterment of the property for railroad purposes. One state, Arkansas, requires that railroads shall be assessed on the same basis as agricultural lands.

Railroad Benefits.—The benefits to a railroad company from the drainage of a tract of land through which its right of way passes may be considered under two general classes, direct benefits and indirect benefits.

Direct benefits are those tangible benefits which result *solely* from the construction of the ditches, drains, levees, and other structures of a district. The laws of Alabama, Delaware, Florida, Georgia, Kentucky, Mississippi (1912), Missouri, Nebraska, North Carolina, Oregon, South Carolina, Virginia, and West Virginia state that the direct benefits to be considered in making the assessment are the "increased physical efficiency and decreased maintenance cost."

The physical efficiency of a railroad is increased if its embankments and track are made more solid and secure, thereby improving the alignment and grade of the track, making greater speed possible, and reducing the danger of derailment. Benefit from this source, however, is not always present; and, if it exists, is extremely difficult to evaluate. If a railroad embankment is so low that the water table is within about 4 feet of the rail, the track is likely to be soft, and the lowering of the water table by drainage is a physical benefit. On the other hand, if the distance from the water table to the rail is greater than about 4 feet, railroad engineers maintain that the railroad is not improved by drainage, except in those cases where the fills slough off regardless of their height. The damage caused by the burrowing of muskrats and other aquatic rodents is generally included in the item of benefit, but such a benefit is usually too trivial to be appreciable.

The decreased maintenance cost may be considered under the following items: maintenance of embankment; additional ballast, life of ties, fence posts, nd trestles; removal or shortening of trestles; and prevention of flood damage. The benefit resulting from decreased cost of maintenance of roadbed and track is frequently overestimated. The testimony of railroad officials in numerous suits shows that the removal of standing water from the right of way does not necessarily decrease the cost of maintenance of track. Where such benefit is present, it is very

difficult to place a monetary value upon it, since the books of the company are not kept in such a way as to show cost of maintenance on individual short stretches of roadbed. In making assessments on these benefits, the commissioners should investigate the situation carefully to make sure that these elements of benefit are present. Where trestles can be replaced in whole or part by fills, the benefit is more definite and more easily estimated, and can be expressed as the capitalized annual saving in maintenance less the cost of making the fills. Likewise, the protection from flood damage is a direct benefit recognized by all railroads. The amount of this benefit is the capitalized average annual loss due to flood damage, as determined from the books of the company. Flood losses include not only the cost of protecting track from high water and repairing damage to roadbed and track, but also the loss of time in operating over dangerous track, loss of cars, freight, and equipment, cost of routing trains over foreign tracks, etc. These losses are more readily estimated than those due to any other item of decreased maintenance.

Indirect benefits are those which are speculative, uncertain, or contingent, such as the increased business to the company resulting from the increase of agricultural production and passenger traffic. The benefit due to increased traffic is so uncertain and indefinite that the courts have almost universally held that assessments must be based on the benefit to the railroad as a property and not to the company in its business. An exception to this rule is found in one of the Mississippi drainage laws which states that increased revenues of a railroad shall be considered in assessing benefits.

Equalization of Railroad Benefits.—An error which is quite common in making assessments against railroads is to assess them to the full amount of the estimated benefits. This is obviously unfair. If the agricultural lands in a proposed district are assessed only a fractional part of the total benefits estimated to accrue to such lands, the railroads should be assessed the same fractional part of their total benefits, and not the full amount of such benefits. Unless this is done, the fundamental principle of all taxation—equality—is violated.

Railroad Damages.—A railroad company is entitled to any damages resulting from drainage district work. The items of damage usually are the cost of building new bridges or rebuilding

old ones over drainage ditches and the cost of removing bridges and track to allow a dredge boat to dig the ditch across the right of way, and of replacing same. The laws of some states specify that the cost of building new bridges or of rebuilding old ones shall be borne by the drainage district; others, that the cost shall be borne by the railroad company. Some states distinguish between bridges over natural water-courses and those over artificial channels. In the former instance, the expense falls on the railroad company; in the latter, upon the district. In most states, the cost of removing bridges to permit the passage of a dredge boat is borne by the drainage district.

Assessments against Highways.—The elements of benefit to a highway from drainage district improvements are the same as those for a railroad, namely, decreased maintenance costs and increased efficiency. There is, however, this difference: Whereas a railroad is financed by private capital, a highway is constructed and maintained by general taxation and there is a question as to whether the benefits are general or special.

The average annual cost of maintaining a highway before the improvement is constructed can generally be determined from the records of the highway commissioners. What the annual cost of maintenance will be after the drainage improvement is completed is difficult to estimate. This will depend on the nature of the soil, the amount of traffic over the improved road, and the completeness of maintenance. Because of the increased traffic over the road and the demand for good maintenance, it may happen that the actual maintenance costs will be increased instead of decreased; but any decrease in benefits on this score is counteracted by the increased benefits due to greater efficiency. The greater use of a highway through a drainage district is evidence of the public need of better roads, and it is but just that the public should pay through general taxation for the benefit it receives. Any decrease in the cost of annual maintenance capitalized at a fair rate of interest is the amount of the total benefit received by the highway under this item.

Increased efficiency is interpreted as meaning the betterment of the highway for traffic purposes. Lowering the water table by drainage eliminates soft spots, makes a drier, smoother, and safer roadway, saves time and tractive effort in hauling over the road, and makes travel over it more comfortable and more desirable. A measure of the benefits from this source is the increased traffic;

but it is difficult to estimate the probable increased traffic over a highway, and hence difficult to estimate the benefits which will accrue to it following drainage.

Because of the difficulty in estimating benefits in a scientific manner and on a logical basis, and because the commissioners desire to make the assessments against the farming land as low as possible, the common procedure has been to assess the highways at as high a valuation as the highway commissioners would agree to, with the result that highways have, in general, been assessed too high.

There is a decided difference of opinion as to who shall pay for the building and maintaining of highway bridges over drainage ditches. In some states, the county pays the cost; in other states, the district pays. A few states specify that the district shall build the bridges and the county shall maintain them.

Assessments against Incorporated Towns.—Here again the laws of the various states differ. In some states, incorporated towns can be included within a district and assessed for drainage benefits; in other states, a drainage district has no jurisdiction over an incorporated town. In the former states, the principal benefits received by a town or city are better health conditions and better outlets for storm-water drainage. Not only can the corporation, as a whole, be taxed, but also individual lots can be assessed for special benefits not accruing to the city as a whole. The assessment against incorporated towns, as a rule, is small and is estimated on a lump-sum basis. The assessment, however, must be based entirely upon benefits due directly to the improvement, and not upon indirect benefits such as the increased prosperity of the landowners within the district, and their increased purchasing power.

Assessments in Two or More Districts.—Agricultural lands, railroads, highways, and incorporated towns may be included in two or more drainage districts and assessed to pay their proportional share of the cost, provided benefits are received from each district.

It frequently happens that the owners of certain contiguous lands in a drainage district desire more minute drainage than that furnished by the district. The law permits these owners to organize a subdistrict for carrying out their plans, and the cost of the work is defrayed by additional assessments on the lands benefited.

If an existing district in whole or in part is benefited by the ditches of a lower district, the land thus benefited can be assessed a second time. In some states the assessment is levied against the first district, as a whole, and the commissioners of that district apportion the assessment against the lands benefited by the new district; in other states, the new district can directly assess any land in the old district which will be benefited.

Conclusions.—The procedure in organizing a drainage district and the methods of making assessments which have been discussed in this chapter are general only, and are given to acquaint the student with the general features of the legal phases of drainage and with the importance of the subject, rather than to make a drainage attorney of him. While the engineer should not attempt to usurp the functions of the drainage attorney, the more he knows of drainage law the better will he be able to serve his clients as an engineer. In fact, at times the engineering and legal phases are so interrelated that the engineer must understand at least the general provisions of the drainage laws of the states in which he practices and the more important decisions of the courts in connection with drainage cases. All drainage districts have legal counsel, and when in doubt the engineer should, by all means, obtain competent advice upon any legal point affecting his work.

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